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VERIFICATION OF COST ESTIMATING PROCEDURES FOR MAPS COMPUTER PR--ETC(U)
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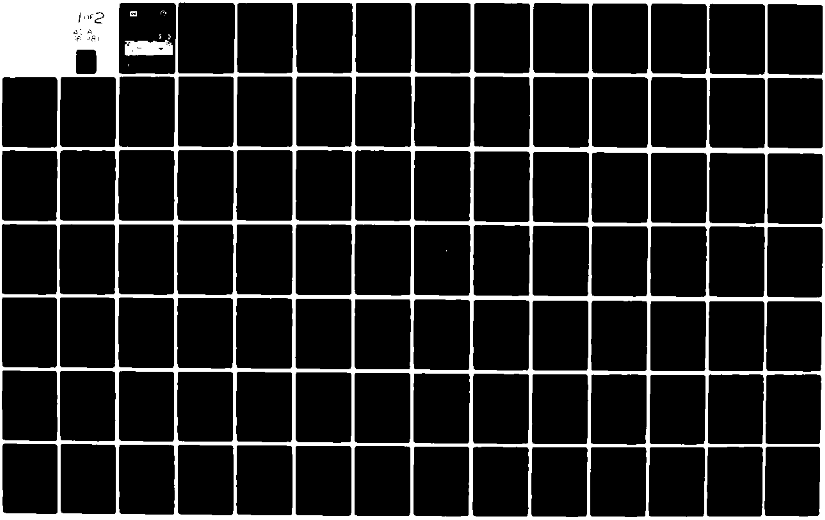

TECHNICAL REPORT EL-82-3

VERIFICATION OF COST ESTIMATING PROCEDURES FOR MAPS COMPUTER PROGRAM

by

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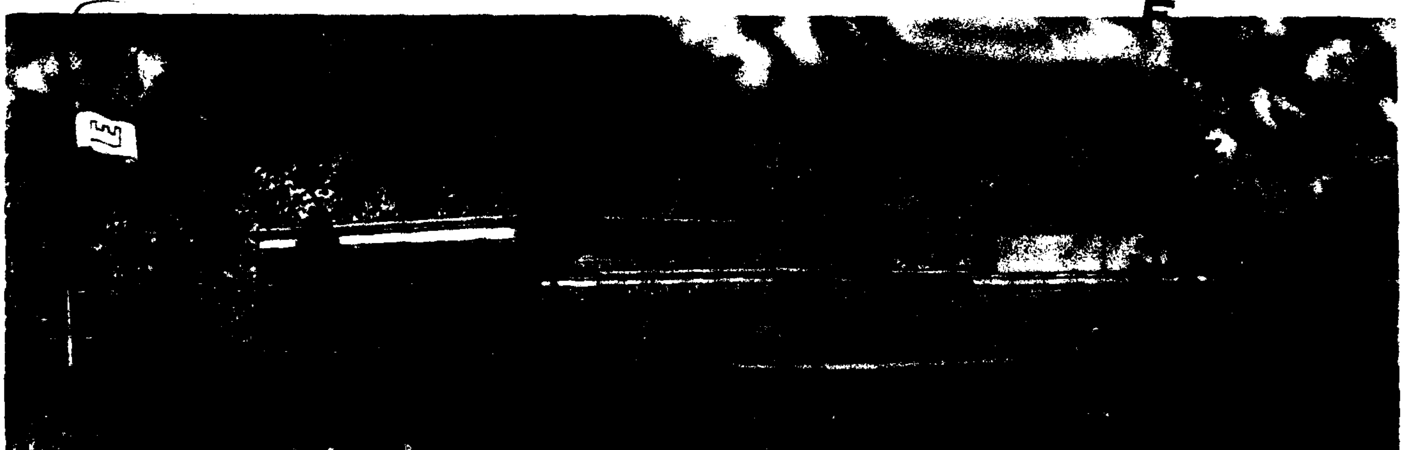
May 1982

Final Report



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The MAPS (Methodology for Areawide Planning Studies) computer program is often used to develop planning level cost estimates for a large array of water resources facilities including dams, pipes, pumping stations, open channels, storage tanks, tunnels, water treatment plants, and wellfields. While the MAPS cost estimates have often been checked against actual facility costs, this study was conducted to systematically verify the MAPS cost functions against a set of

(Continued)

20. ABSTRACT (Continued).

actual costs of projects. Data were provided for the study by a large engineering consulting firm. The study showed that while some minor modifications to the program were required, the MAPS estimates were of sufficient accuracy for planning studies. For the 35 facilities considered, the geometric mean of the percent differences between MAPS estimates and actual costs was 13.9 percent.

PREFACE

The MAPS (Methodology of Areawide Planning Studies) computer program was developed at the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., under the Water-Based Disposal Subprogram of the Wastewater Management Program. This verification study was conducted under the MAPS work unit (CWIS No. 31572) of the Water Conservation and Supply Program.

The study was conducted by Ms. Anita K. Lindsey and Dr. Thomas M. Walski of the Water Resources Engineering Group (WREG), Environmental Engineering Division (EED), Environmental Laboratory (EL), WES. Data for the study were provided by the Gainesville, Fla., office of the CH2M-Hill consulting firm under Purchase Order DACW39-81-M-0722. The project manager for CH2M-Hill was Mr. Stephen Hahn. The work unit technical monitor at the Office, Chief of Engineers, was Mr. James Ballif (DAEN-CWE-BU).

The study was conducted under the direct supervision of Mr. Michael R. Palermo, Chief, WREG, and under the general supervision of Mr. Andrew J. Green, Chief, EED, and Dr. John Harrison, Chief, EL.

The Commander and Director of WES was COL Tilford C. Creel, CE. The Technical Director of WES was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acres	4047	square metres
acre - feet	1233	cubic metres
cubic feet	0.02832	cubic metres
cubic feet per second	0.02832	cubic meters per second
cubic yards	0.7645	cubic metres
feet	0.3048	metres
feet per second	0.3048	metres per second
foot of water (39.2°F)	2989	pascals
gallons (U. S. liquid) per minute	0.003785	cubic metres per minute
inches	2.540	centimetres
miles (U. S. statute)	1.609	kilometres
million gallons (U. S. liquid)	3785	cubic metres
million gallons (U. S. liquid) per day	0.04381	cubic metres per second
pounds per square inch	6894	pascals
square feet	0.09290	square metres

VERIFICATION OF COST ESTIMATING PROCEDURES FOR MAPS COMPUTER PROGRAM

PART I: INTRODUCTION

Background

1-1. The MAPS (Methodology for Areawide Planning Studies) computer program is a multipurpose program developed at the U. S. Army Engineer Waterways Experiment Station (WES) for use in planning level water resource studies. The program is most commonly used to make cost estimates for comparisons of many typical facilities (referred to as "modules" in MAPS) such as dams, force mains, pump stations, open channels, storage tanks, tunnels, water treatment plants, and wellfields. Additional capabilities include preliminary design, simulation, and economic analysis.

1-2. The cost functions contained in the MAPS design modules have been synthesized using a large array of the most up-to-date cost data available. Each time the program has been used in a study, the program developers at WES have encouraged the users to check the MAPS estimates against actual costs of facilities in the study area to ensure that the calculated cost estimates are appropriate for the study. Therefore, the program has been independently checked by several Corps of Engineers (CE) Districts and their consultants and, with a few exceptions, has been found to be sufficiently accurate for planning studies.

1-3. Nevertheless, a systematic study has never been conducted to verify the MAPS cost estimates against cost data not used in the initial development of the program. This type of verification is usually required for most computer programs whether they be hydraulic, economic, or environmental models.

Purpose

1-4. The purpose of this study is to verify the MAPS cost estimating procedure against an independently determined set of cost data. From the analysis it will be possible to: (a) determine the accuracy of the individual modules, (b) identify and correct minor shortcomings of the program, and (c) identify potential program modifications and additions.

1-5. This report has been prepared to present the results of the verification study, thereby providing MAPS additional credibility with both planners and estimators. In addition, readers should gain a better appreciation of the problems associated with planning level cost estimation and a better understanding of the accuracy of the resulting cost estimates.

1-6. This report is not intended to be a primer on MAPS and, as such, is written for an audience that is already familiar with the program. For those not already familiar with MAPS, Appendix A has been included to provide the reader with an overview of the capability of the program, and Appendix B has been included to describe the philosophy used in developing the MAPS cost estimating procedures.

Approach

1-7. The approach used to conduct the study can be divided into five steps:

- a. Collect design and cost data for individual projects.
- b. Make cost estimates with MAPS.
- c. Make initial cost comparisons.
- d. Adjust design data to correct problems with initial estimate and rerun MAPS.
- e. Make final comparisons.

Each of these steps is described in more detail in the following paragraphs.

1-8. It was felt that the best data could be obtained from an engineering firm with considerable experience in designing a wide variety of water resources projects. Data were purchased from the firm of CH2M-Hill, which will be referred to for the remainder of the report as "the contractor." The point of contact with the contractor was the firm's Gainesville, Fla., office, but data were supplied from projects throughout the country. The contractor provided two types of data: (a) design parameters required as input for MAPS, and (b) actual costs of projects for verification purposes. It most cases, data were provided for five projects for each module. The exceptions to this were open channels and tunnels where the contractor did not have adequate data, and pump stations where data for one of the facilities were discarded due to inconsistencies.

1-9. Initially, the data were entered into the MAPS program. Where the data were not complete, MAPS default values were used. Unit price data for individual items were entered whenever they were available, although in many cases the MAPS estimates of unit prices were used. The program was then run for each facility.

1-10. Cost estimates for these initial runs were compared with cost data provided by the contractor based on the costs in year-of-construction dollars. (Costs were later updated to January 1980 dollars, using appropriate cost indices, for display in figures on a consistent basis.) Where bids for a given project were available, the low bid was used as the "actual" price. In a few cases in which the low bid was significantly lower than the engineer's estimate and the other bids, the engineer's estimate, based on detailed plans and specifications, was used. Where bid tabulations were not available, the engineer's estimate, as opposed to the actual cost, was used. Comparisons were made solely on the basis of construction costs. Initially, it was hoped that adequate data would be available to verify MAPS operations and maintenance (O&M) cost estimates. However, because of the manner in which utilities generally keep O&M cost records, it was not possible to determine O&M costs for individual facilities or components; therefore, these comparisons were not made.

1-11. In most cases, the initial MAPS cost estimates were not sufficiently close to the actual costs to be acceptable. There were two reasons for this. The first involved inadequacies in the data and/or special design problems. For example, the actual construction costs of water treatment plants included the cost of intake structures, which are not considered as part of the design by MAPS. Also, no note was made of the fact that special drilling equipment was required for one of the wells. In these instances, the input to MAPS was adjusted to account for the special condition or the cost of special facilities (e.g., intakes) were added to the MAPS estimate. In some cases, the costs were not adjusted because the MAPS user in a planning study would not have access to the data to adjust the costs; hence, the adjustment would not result in a correct reflection of the accuracy of MAPS.

1-12. The second source of error in the initial estimate existed in MAPS itself. This could be attributed to three causes: (a) programming errors, (b) limited range of cost functions, or (c) an unsuitable function. The few programming errors that were found were immediately corrected. In some instances, a cost function was found to be appropriate only for a limited range of sizes or types of facilities. For example, the cost of siphons in canals was found to be good only for large siphons, so additional data were used to extend the range to flows as low as 1 cfs.* In another instance, the wellfield piping cost algorithm, which was only appropriate for wells arranged in a circle, was modified to account also for wells arranged in a line. Finally, where the cost function was found to be weak, it was replaced using additional cost data. With only a few exceptions, where data were very scarce, the cost data from the verification study were not used in modifying the cost functions.

1-13. Once all adjustments were made, a final run of the MAPS program for each module was made. The results of these runs are presented in the body of this report.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) is presented on page 6.

Overview

1-14. The following nine parts in this report contain the module-by-module results of the verification study. In each part, the results of the verification of MAPS costs against the contractors data are presented first. Next, the MAPS costs are compared with other sources of cost data in the literature to illustrate either that the MAPS functions are consistent with the literature or how and why they differ. Modifications made to the program as a result of this study are then presented.

1-15. As stated earlier, Appendices A and B contain references for those wishing to know more about MAPS and cost estimating procedures in MAPS. If these do not contain an adequate level of detail, the reader is referred to the MAPS User's Guide and Documentation, Engineering Manual EM 1110-2-502 (Office, Chief of Engineers (OCE) 1980).

1-16. Appendix C contains supplements to the MAPS User's Guide and Documentation developed as a result of this study. Since the next complete revision of the MAPS manual is scheduled for 1986, the reader is encouraged to save this Appendix as it is the only documentation of the revisions until the completely revised manual is published.

1-17. Not all of the MAPS modules were addressed during this study. Modules such as headwaters or service areas were not considered since they are only used for simulations, not cost estimating. The reservoir module was not considered since it has been superceded by the dam module. The gravity main (i.e., pipes not flowing full) module was not considered since it is outdated. (Note that pipes which flow full, whether by gravity or pumping, are addressed in the force main module, and tunnels, not flowing full, can be considered in the tunnel module.)

Accuracy

1-18. In evaluating the MAPS cost estimating procedure, the question that must be asked is "How accurate should cost estimates in planning studies be?" There is no simple answer to this in the CE regulations or manuals. For government estimates based on detailed plans and specifications, the regulation on engineering contracts (ER 1180-1-1; OCE 1969) requires that for Civil Works projects all bids be rejected if the low bid is more than 25 percent higher than the government estimate without profit. Certainly, a planning level tool such as MAPS should not be required to be more accurate than a government estimate based on detailed plans and specifications. It is not uncommon for bids on a given project to vary by as much as 50 percent.

1-19. The cost estimating manual (EM 1110-2-1301) does not give expected accuracy for planning level estimates. It does state that for small (<$10 million) projects in the survey and review stage, 25 percent should normally be allowed for contingencies.

1-20. EM 1110-2-1301 does state, "The degree of accuracy and precision in estimates at various stages of design will be considered in light of the use thereof, such as comparison and elimination of alternatives, weeding out of less practicable solutions, etc." It is important to remember that the principal use of estimates in planning studies is for comparison of alternatives. Since the estimating procedures in MAPS are internally consistent, the program will be serving its purpose in that relative costs will be accurate even if there are some inaccuracies in absolute costs.

1-21. In light of the above discussion, the MAPS estimates should be considered accurate if they are within 25 percent of the actual cost as long as the estimate used in selecting the recommended plan is prepared in as much detail as possible, and corrections are made to the estimates for any extraordinary conditions not accounted for by MAPS.

PART II: DAMS

Introduction

2-1. The MAPS dam module calculates the cost of a dam and reservoir given a description of the dam and ground elevations at the damsite. The cost of a concrete dam is based on the volume of concrete (determined by average end area method) and the volume of required stripping. The cost of earthfill dams is based on the cost of stripping, foundation trench, toe drains, embankment protection, and the price of excavating, hauling, placing, and compacting pervious and impervious material. Costs for spillways and outlet works are given as a function of head and flow. Spillway gates and bridges may also be specified. Relocations of primary and secondary roads, railroads, and power lines may be accounted for by specifying the length and type of these items. The program does not calculate costs for items such as fish ladders and recreation facilities. Costs of this type must be combined and entered as miscellaneous costs. A contingency cost may also be specified or assumed by MAPS as 15 percent of the construction cost. Unit prices for items such as common and impervious material, concrete, and riprap may also be input by the user or determined by MAPS based on required volumes.

Input Data

2-2. Data were provided by the contractor for two earthfill dams with earth spillway sections, an earth dam with a concrete spillway section, an earth dam with a spillway located separate from the dam, and a diversion dam. These dams were constructed in California, Oregon, Nebraska, and Colorado between 1973 and 1979. Table 2-1 lists some of the significant characteristics of each dam, the actual construction costs, and the MAPS cost estimates. The results are shown in Figure 2-1.

Table 2-1

Comparison of Actual and MAPS Costs for Reservoirs

Case	Description	Construction Cost, $ (1980 dollars) Actual	MAPS
I	Earthfill Dam Concrete spillway section Ungated spillway Spillway capacity: 20,000 cfs Outlet capacity: 250 cfs Storage volume: 3,000 acre-ft Embankment protection upstream Maximum ground to crest distance: 76 ft Year: 1973	2,192,649	1,890,000
II	Earthfill Dam Spillway separate Ungated spillway Spillway capacity: 1,800 cfs Storage volume: 35,000 acre-ft Embankment protection upstream Maximum ground to crest distance: 101.6 ft Year: 1976	1,597,071	2,240,000
III	Earthfill Dam Earth spillway section Ungated spillway Spillway capacity: 14,000 cfs Storage volume: 2,160 acre-ft No embankment protection Maximum ground to crest distance: 61.5 ft Year: 1973	503,985	601,000
IV	Earthfill Dam Spillway separate 2 spillway gates Spillway capacity: 35,000 cfs Outlet capacity: 800 cfs Storage volume: 300,000 acre-ft Embankment protection upstream Maximum ground to crest distance: 217 ft Year: 1973	7,541,143	8,540,000
V	Concrete Dam Concrete spillway 1 spillway gate Spillway capacity: 2,000 cfs Outlet capacity: 100 cfs Maximum ground to crest distance: 17.5 ft Year: 1979	1,224,395	1,110,000

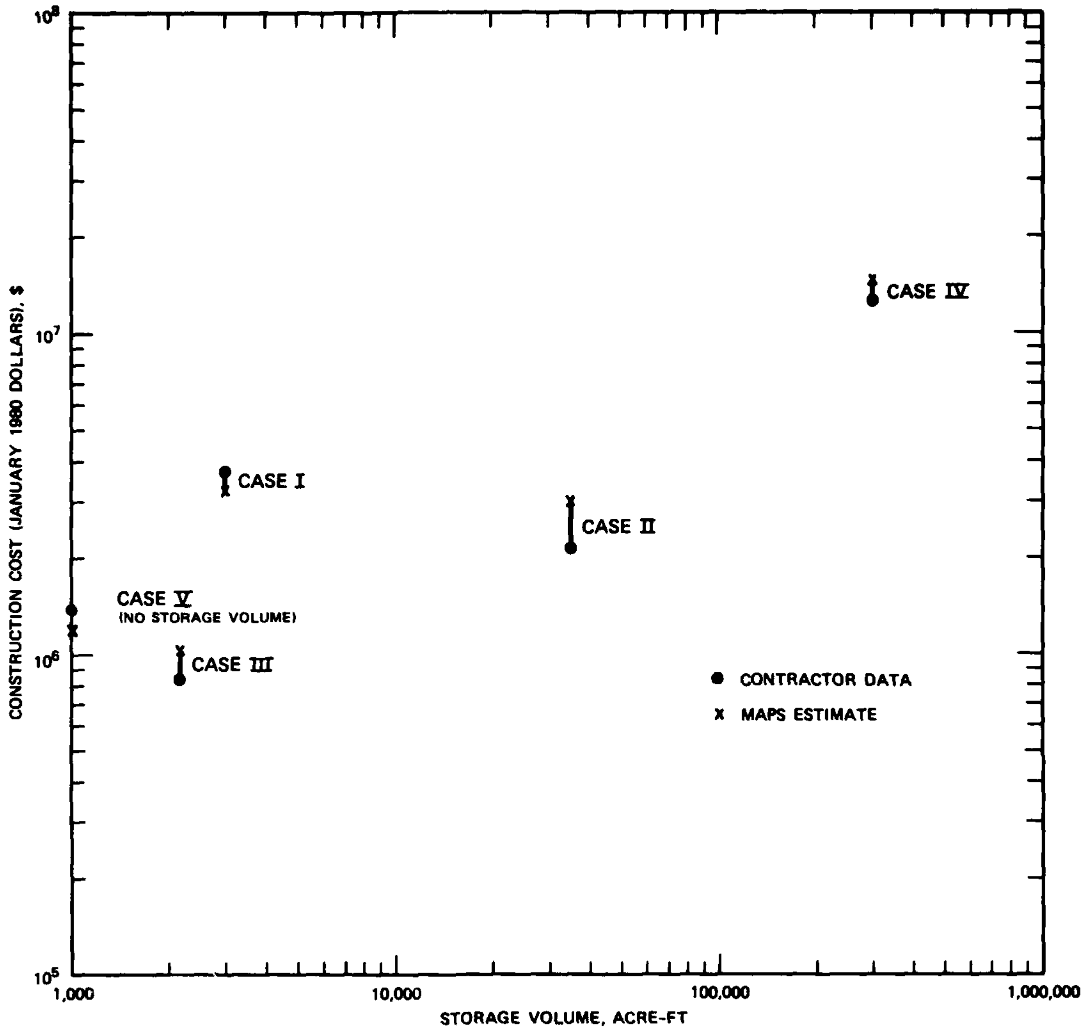

Figure 2-1. Summary of reservoir cost estimates

Discussion of Results

2-3. Costs for an earthfill dam with an ungated concrete spillway section are presented in Case I (Table 2-1). The contractor provided a bid tabulation for this project, so it was possible to make comparisons of individual items. Predicted costs for the embankment are within 6 percent of the actual bid cost. (Unit prices for pervious and impervious material were input to the program, rather than having the program calculate default costs.) MAPS total estimated construction cost is less than 14 percent lower than the actual cost. The total includes relocation costs for structures and utility poles and costs for canals, both of which were input to MAPS as lump sums based on the contractor's data.

2-4. The spillway for the dam in Case II is located in a "saddle" in the reservoir perimeter rather than at the dam. MAPS estimated riprap cost is much higher than the actual cost due to the irregular valley shape at the damsite. Figure 2-2 shows the configuration of the dam, the area of actual embankment protection, and the area assumed by MAPS to be protected based on the program input. If cost based on the actual volume of riprap is used instead of the cost determined by MAPS, the new total construction cost is within 16 percent of the actual cost.

2-5. Costs for an earthfill dam with an earth-lined spillway are given in Case III. No embankment protection is included in this case because material used to construct the upstream embankment section contains a large percentage by volume of coarse gravel. MAPS total estimated construction cost is approximately 19 percent higher than the low bidder for the project.

2-6. The spillway for the dam in Case IV is a gated "chute" type spillway located in a rock cut on the abutment. Relocations for this project involved a 4-mile stretch of secondary highway. Miscellaneous costs were directly input to the program to account for a boat ramp. The difference in the actual and estimated total construction cost is approximately 13 percent.

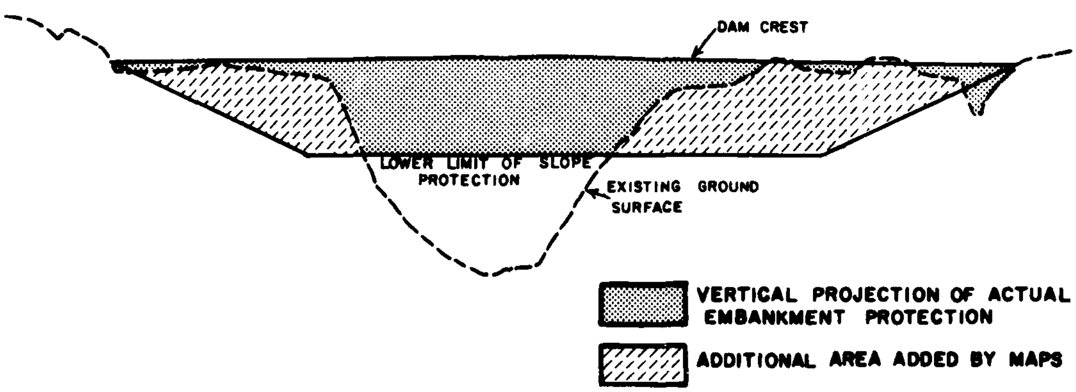

Figure 2-2. Profile of dam embankment for Case II

2-7. The dam considered in Case V is a diversion structure on a canal system. The total length of the dam is the sum of the lengths of three different cross-sectional shapes: (a) an ogee-shaped concrete overflow section, (b) a "gated" section approximately 15 ft wide, and (c) a 1-ft-wide vertical concrete wall. Approximately half of the vertical wall is exposed, while the remainder is buried in the left abutment as a cutoff wall. In applying MAPS to this situation, the dimensions of the concrete overflow section were used for input data, as about 70 percent of the complete structure consisted of this cross-sectional type. The ogee shape was approximated as illustrated in Figure 2-3. For initial runs, MAPS estimated total cost was considerably lower than the actual cost. This was due primarily to the fact that the unit price override used for concrete was based on mass concrete only when, in reality, over half of the concrete used was structural concrete with a much higher unit price. Since only one unit price may be entered in the program, a weighted average was used. With this adjustment, there was less than a 10-percent difference between actual and estimated total costs.

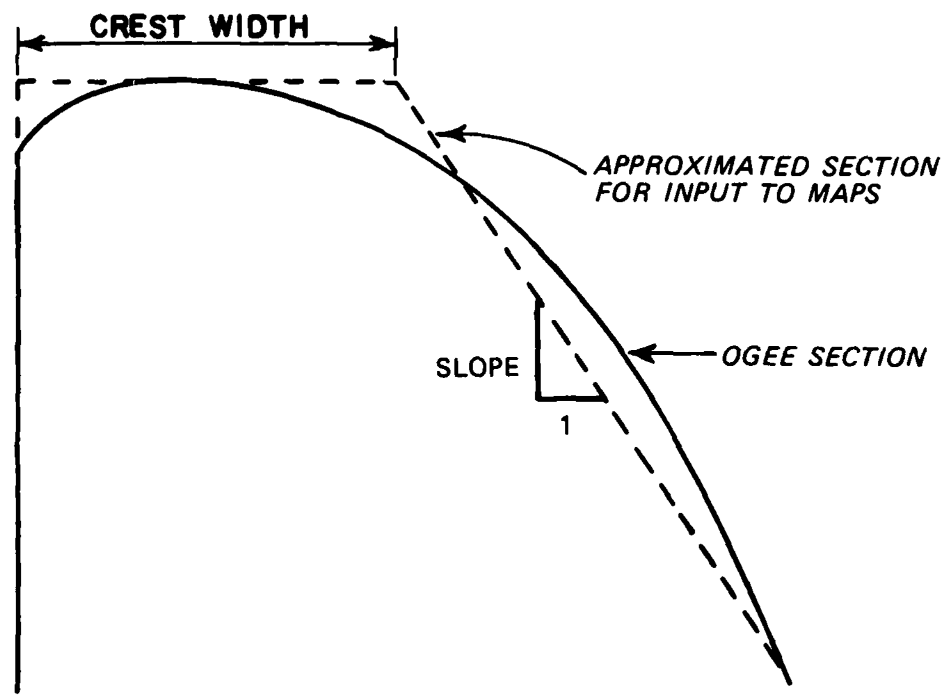

Figure 2-3. Approximation of ogee-shaped crest for MAPS

Additional Verification

2-8. Many procedures developed in the past to estimate reservoir costs sought to relate reservoir cost only to storage volume. While this is useful for gross estimates, the large number of variables in reservoir design renders the use of such a simple method unrealistic. Figure 2-4 gives a comparison of actual costs and MAPS estimates of the projects studied, and functions developed by Koenig (1966) and Dawes (1970) which estimate cost as a function of storage volume. (Case V is not included in this comparison since storage volume is not an applicable parameter for a diversion dam from a canal). Dawes' function gives a

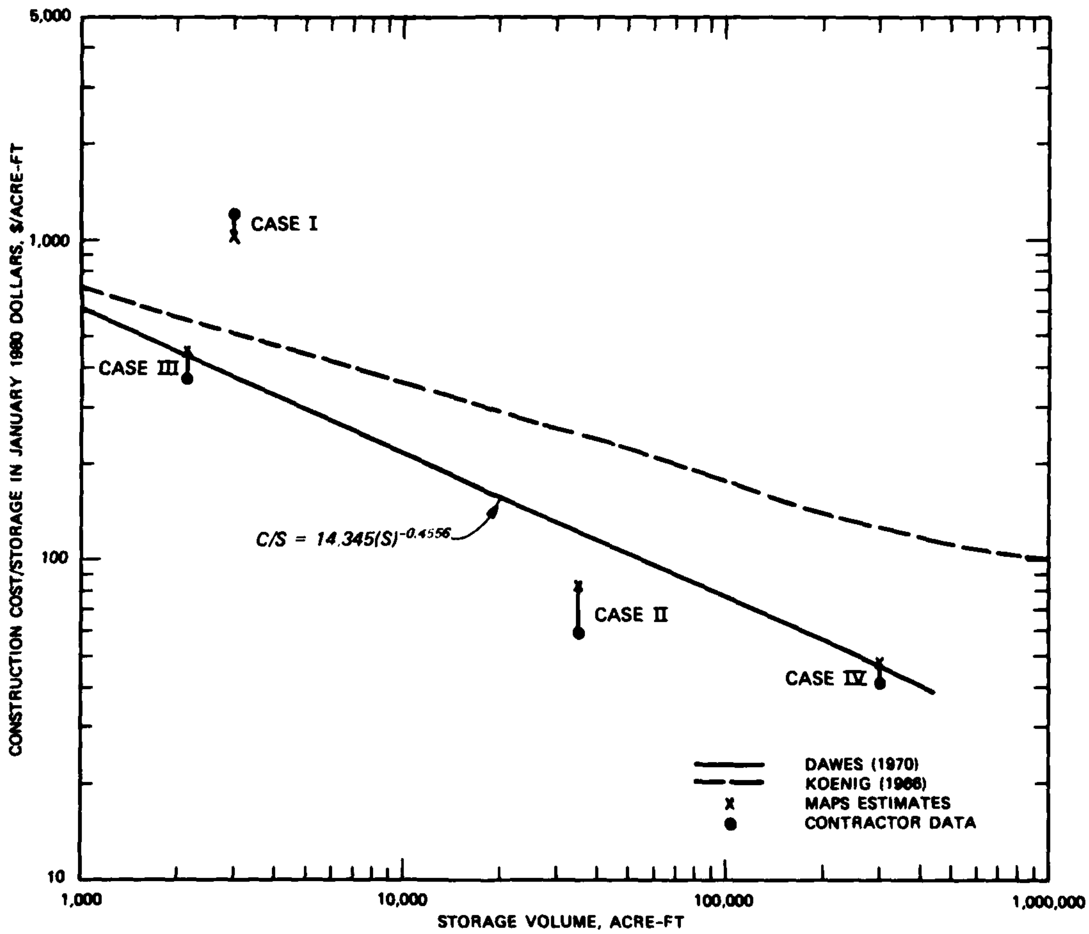

Figure 2-4. Comparison of reservoir costs as a function of storage

good indication of the cost for three of the cases investigated. However, the MAPS estimate provides accuracy as good as or better in every case due to the larger number of variables accounted for in the program.

Program Modifications

2-9. The original function in MAPS used to determine the cost of spillways for earth dams was based on data from the Bureau of Reclamation (1959) for dams constructed between 1954 and 1966. Data were obtained for more recent spillway construction as part of the High Plains Ogallala Aquifer Study. Data from the Corps were obtained as well. An updated function based on these data provided much better accuracy for comparisons with contractor data in this study.

2-10. MAPS users have noted in the past that volumes of riprap are often overestimated by the program. The original function calculated volumes as crest length × distance from crest to lowest point of embankment protection × thickness of riprap. This equation best applies to valley shapes tending toward a rectangular shape. In reality, it is more common for the valley to taper in toward the center of the dam when going from the crest to the bottom of the dam. Therefore, the equation has been modified to more accurately represent the latter situation as shown in Figure 2-5. (Note: the example in the figure is based on the valley shape in Case I.)

Summary

2-11. Obtaining a reasonable cost estimate for reservoirs requires knowledge on the part of the planner of a great many variables, including topography of the damsite, cross-sectional configuration of the dam, dam crest and spillway elevations, etc. The results of this verification study show that good estimates may be obtained using MAPS, provided the user is familiar with the assumptions on which individual functions are based. For example, as mentioned earlier in this chapter concerning the dam in Case II, knowledge of the method provided in MAPS for

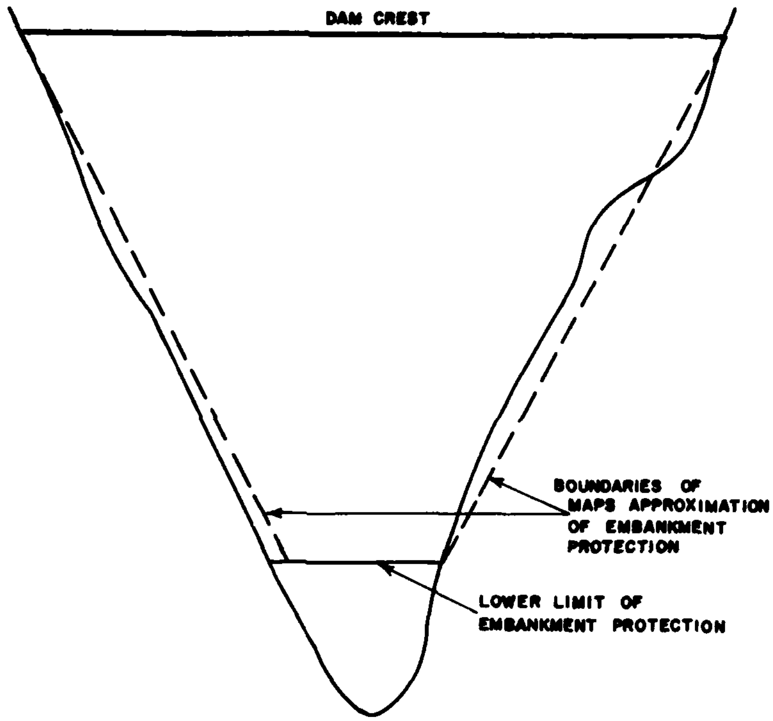

Figure 2-5. New method for estimating embankment protection requirements

calculating area of embankment protection allows the user to determine the applicability of the results to a particular situation. Adjustments may then be made by the user for unusual cases.

2-12. Based on this study, it appears that the MAPS estimate is also applicable to diversion dams, provided an approximate ratio of structural concrete to mass concrete is known. However, further testing should be conducted before this can definitely be concluded for all diversion dams.

PART III: FORCE MAINS

Introduction

3-1. The MAPS force main module calculates costs for the construction of any pipeline designed to flow full. It also calculates either (a) the diameter and required head, given the flow; (b) the flow and diameter, given the available head if pumping is not required; or (c) the head required, given the diameter and flow. Major elements in force main construction costs include:

a. Pipe cost. The cost of purchasing, hauling, and laying pipe is a function of pipe material, diameter, length, and maximum pressure.

b. Excavation and backfill. The cost of excavation and backfill depends on the dimensions of the trench and the type of material. Concrete bedding may be specified if required. Trench dimensions may be specified by the user or assumed by MAPS.

c. Appurtenances. The number of valves, bends, and hydrants may be specified by the user. Costs for valves and bends are based on pipe diameter, and hydrant costs are based on a "standard" hydrant on a 6-in. line.

Input Data

3-2. Actual design and cost data were obtained from the contractor for five force mains constructed since 1972. The type of piping, length, diameter, and year of construction of each force main are presented in Table 3-1.

Discussion of Results

3-3. Actual costs for ductile iron pipelines are given in Cases I and III. MAPS estimates are 17 percent higher in CASE I and 33 percent higher in Case III. The project engineer provided additional information as follows:

Table 3-1

Comparison of Actual and Estimated Costs for Force Mains

Case	Description	Construction Cost, $ (1980 dollars) Actual	MAPS
I	Peak flow: 15 mgd Diameter: 48 in. Length: 30,000 ft Ductile iron pipe Year: 1979	2,800,000	3,281,000
II	Peak flow: 130 mgd Diameter: 84 in. Length: 74,500 ft Prestressed cylinder pipe Year: 1973	11,898,000	11,270,000
III	Peak flow: 12.5 mgd Diameter: 30 in. Length: 25,500 ft Ductile iron pipe Year: 1975	1,102,360	1,464,000
IV	Peak flow: 55 mgd Diameter: 36 in. Length: 2,200 ft Prestressed cylinder pipe Year: 1972	208,827	122,200
V	Peak flow: 20 mgd Diameter: 36 in. Length: 26,750 ft Prestressed cylinder pipe Year: 1976	1,452,000	1,609,000

a. Bidding for the projects was highly competitive.

b. No relocations were required.

3-4. Prestressed cylinder pipe was used in the pipelines for Cases II, IV, and V. MAPS estimates for Cases II and V are, on the average, within 8 percent of the actual construction costs. The actual cost for Case IV, however, is considerably higher than the estimate. The project engineer suggested that this may be due to the large quantity of portland cement paving that was required, as well as the fact that the pipeline was laid in a highly congested industrial area. Extensive dewatering was also required. Data were obtained from the contractor for a gravity main included in this same project. The actual costs were unusually high for all items in this particular project.

3-5. Figure 3-1 provides a summary of the results of the construction cost comparison.

Additional Verification

3-6. Figure 3-2 shows a comparison of MAPS estimates to costs determined in a study for the U. S. Army Engineer Division, New England (1977) for ductile iron force mains. The MAPS estimates for Cases I and III (ductile iron pipe) show a close correlation with the New England costs. The total costs include excavation and backfill, bedding, laying labor, and valves and fittings.

3-7. Figure 3-3 gives a comparison of total construction costs for prestressed concrete force mains. Dickson (1978) compiled costs for various types of transmission mains constructed in the southwestern United States. The costs given for prestressed concrete pipe were all for 54-in.-diam pipes, and a range is shown based on these data. The New England Division (1977) developed a cost curve for a range of diameters from 36 to 96 in. Also, cost functions developed by the West River Aqueduct Study Management Team (1978) are presented for a range of pressures from 200 to 900 ft. MAPS estimates calculated for Cases II, IV, and V fall between the cost curves from the other sources.

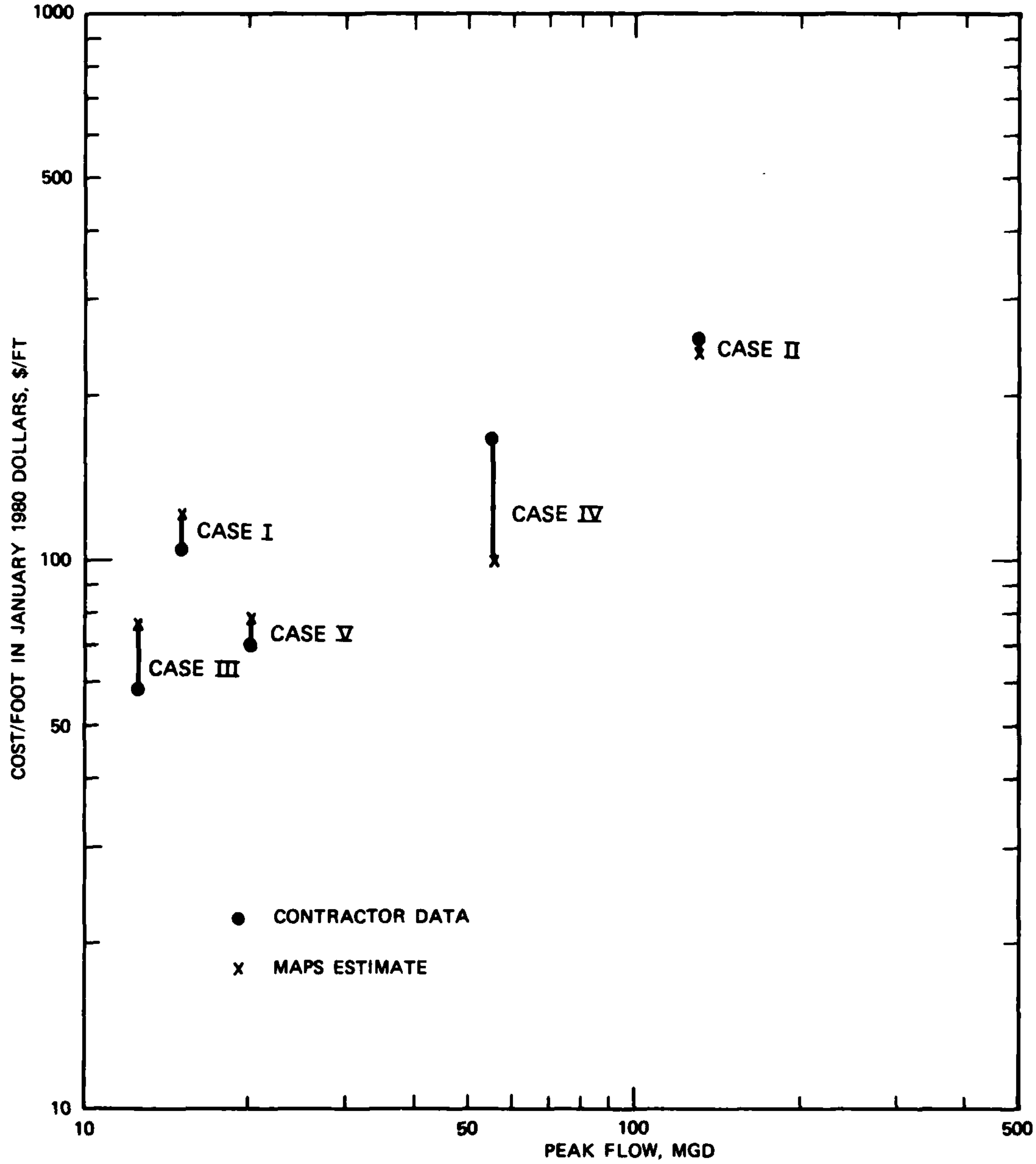

Figure 3-1. Comparison of force main construction costs

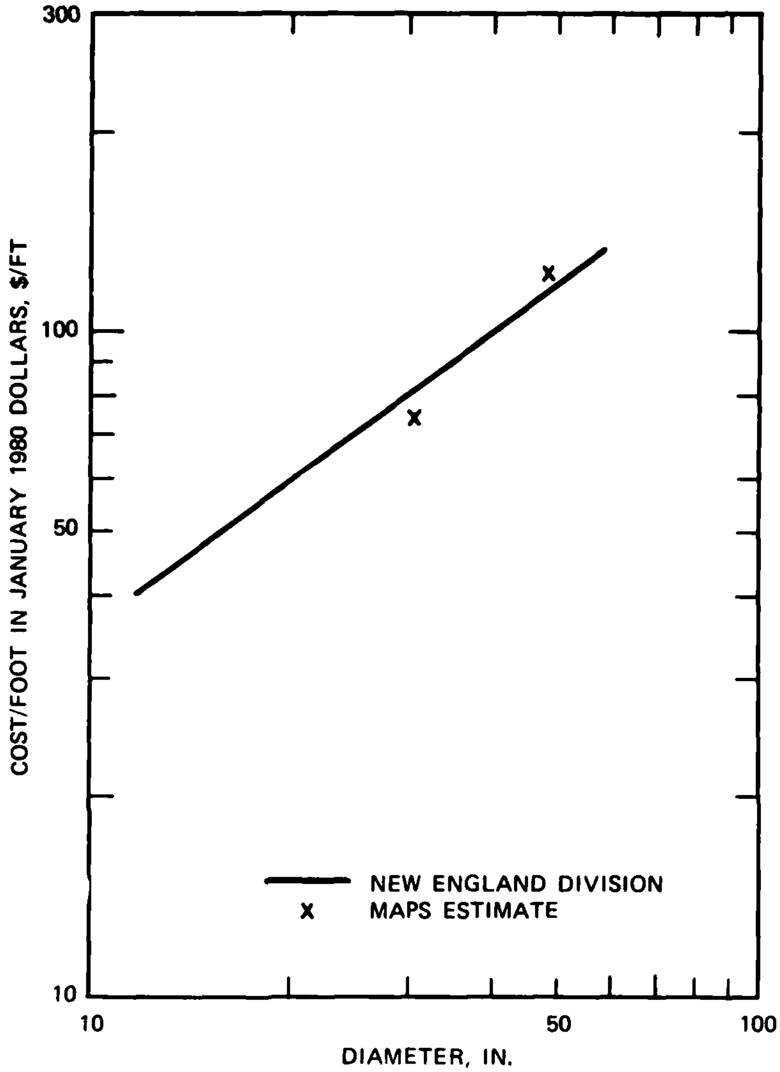

Figure 3-2. Construction cost for ductile iron force mains

Program Modifications

3-8. The culture multipliers previously used in MAPS to account for the change in cost with varying site conditions (open country, residential, etc.) were based on the Louis Koenig Research, Inc. (1974), study. A more recent study for the U. S. Environmental Protection Agency (Dames & Moore 1978) resulted in the development of a set of modifiers

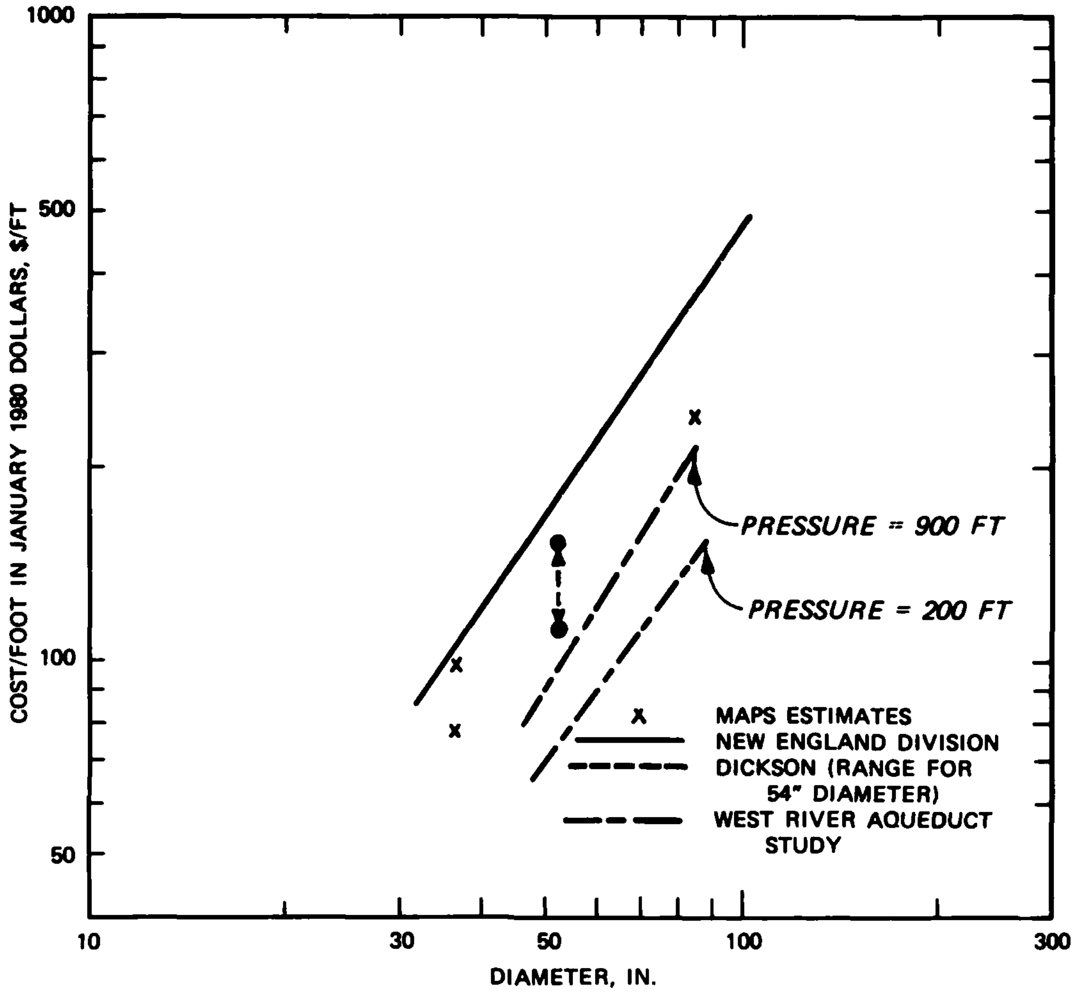

Figure 3-3. Construction cost for prestressed concrete force mains

based on data from 455 facilities. Applying these cultural modifiers to the projects investigated in this study resulted in cost estimates closer to the actual costs than using Koenig's values; therefore, MAPS has been updated to include these modifiers. If the type of terrain is not specified when running MAPS, the program assumes a value of 1.0 for the multiplier.

3-9. The functions used to determine the weight of reinforcing steel required for reinforced concrete pipe, prestressed cylinder pipe, and pretensioned cylinder pipe have been modified based on a more exact curve fit for the original data used to develop the equations.

3-10. Figure 3-4 shows trench bottom widths for various pipe diameters based on data from Means (1979) and the Cast Iron Pipe Research Association (1976). The default value for trench width for each pipe size has been modified in MAPS based on these data. The user may still specify this value if desired.

3-11. Figure 3-5 gives updated excavation unit prices as a function of trench depth based on data from Means (1979). The MAPS default unit prices have been modified to more closely approximate these data.

Summary

3-12. Force main construction costs are highly dependent on the

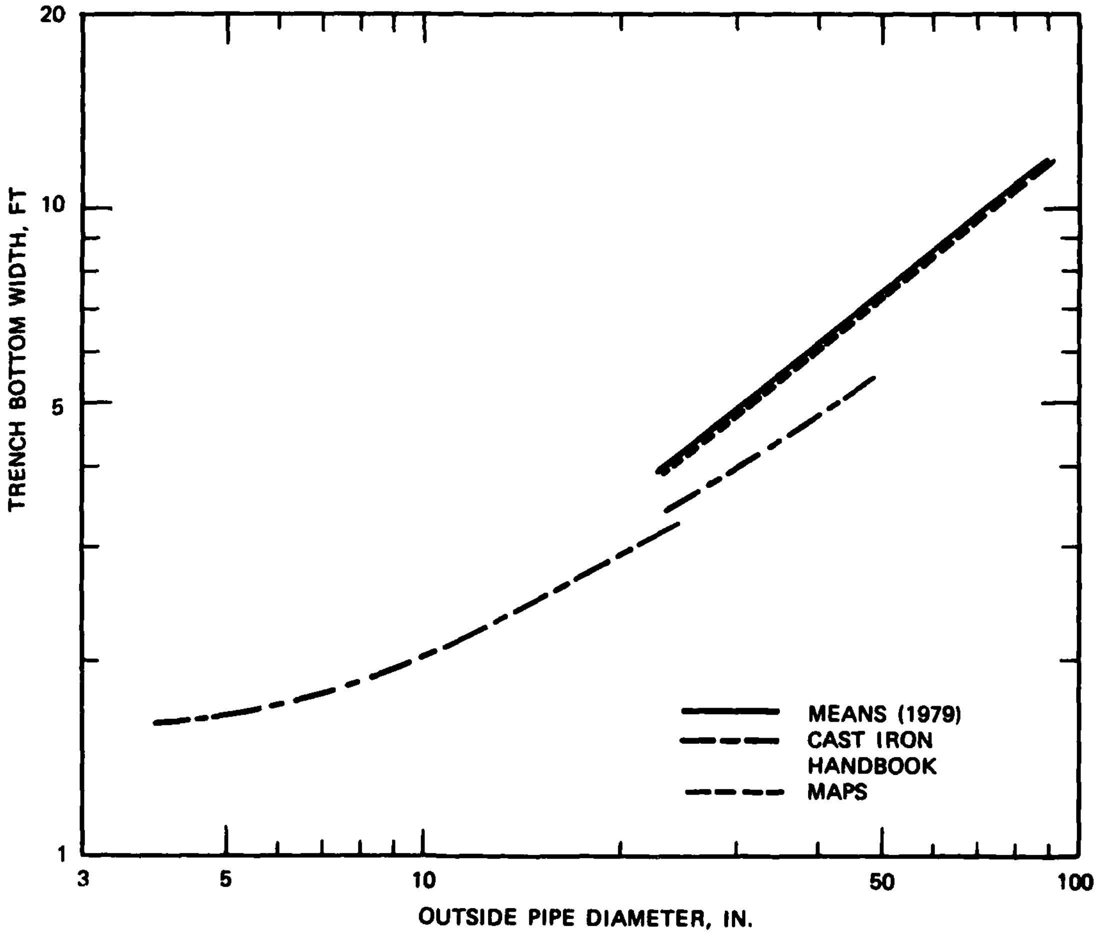

Figure 3-4. Force main trench width as a function of pipe diameter

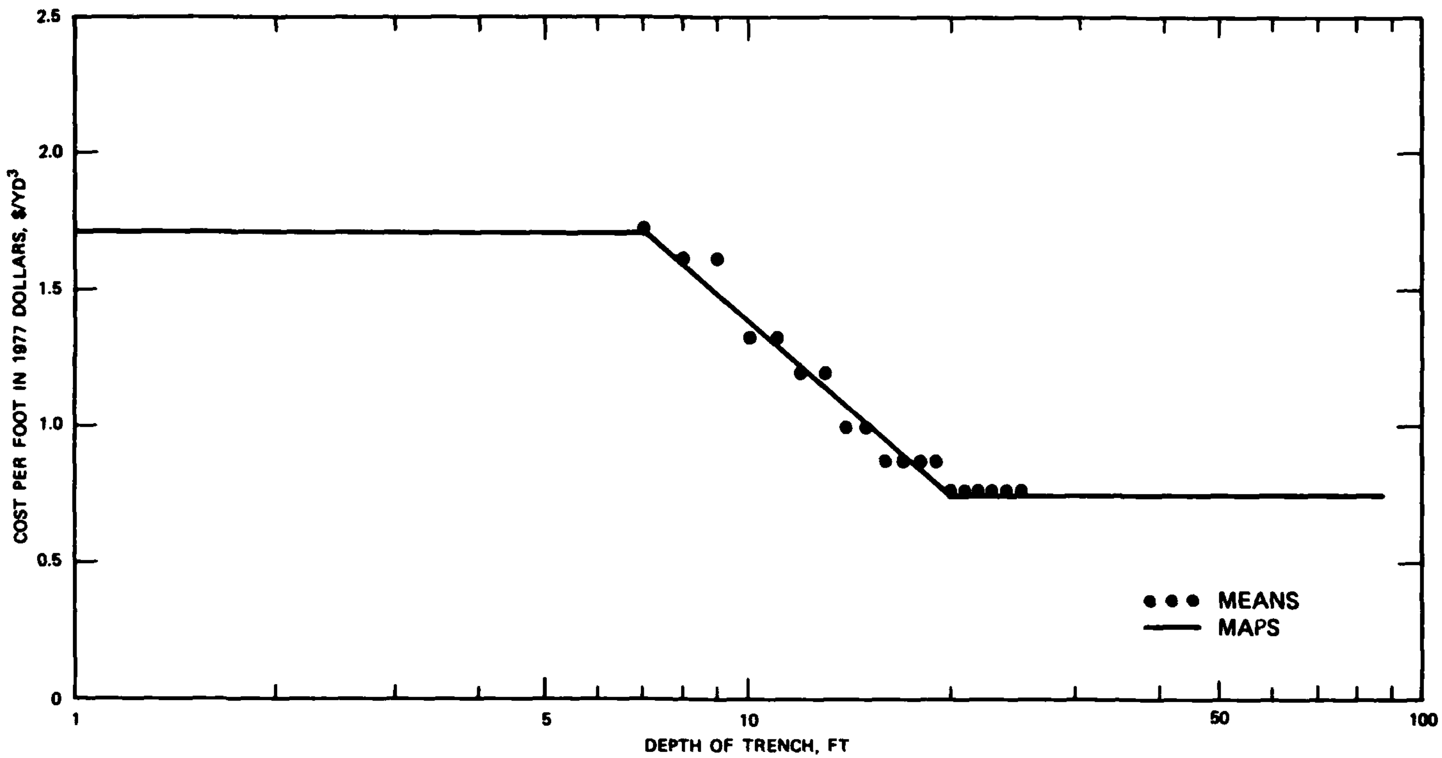

Figure 3-5. Excavation costs

level of bidding competition for a particular project as well as local site conditions. The results of this verification study show that MAPS estimates are valid for the two types of pipe for which data were obtained; the results were confirmed by several additional sources.

PART IV: PUMP STATIONS

Introduction

4-1. Pump station construction costs are estimated by the MAPS program based primarily on peak flow rate, required head, structure complexity, and wet well volume (if required). Costs are categorized by MAPS as mechanical, electrical, structural, switchyard (optional), and wet well costs.

Input Data

4-2. General design characteristics and actual construction costs for four pump stations constructed since 1975 were reviewed for this comparison study. Table 4-1 provides a brief description of each station, the actual construction costs, and the MAPS predicted costs.

Discussion of Results

4-3. A detailed breakdown of actual construction costs was obtained for the pump station in Case I. Individual cost items were organized for comparison with the MAPS categories as closely as possible, and are compared item by item below:

Item	Construction Costs, $ Actual	Construction Costs, $ MAPS
Total construction cost	1,632,860	1,865,939
Mechanical equipment	398,400	439,000
Electrical equipment	75,100	120,227
Structure and wet well	460,860	424,215
Miscellaneous equipment (piping, manifolds, valves, etc.)	698,500	451,896
Contingencies (30%)	--	430,601

Overall, the actual total construction cost is less than 13 percent lower than MAPS predicted cost.

4-4. The pump station in Case II is classified by MAPS as a small pump station (flow less than 5.0 mgd). Structural, mechanical, and

Table 4-1

Comparison of Actual and MAPS Costs for Pump Stations

Case	Description	Construction Costs, $ Actual	Construction Costs, $ MAPS
I	Treated Water Pump Station Maximum flow: 160 mgd Head required: 35 ft Improved structure Wet well volume: 0.2 mg No switchyard Year: 1978	1,632,860	1,865,939
II	Wastewater Pump Station Maximum flow: 1.14 mgd Head required: 58 ft Improved structure Wet well volume: 0.006 mg No switchyard Year: 1979	122,000	116,500
III	Wastewater Pump Station Maximum flow: 12.5 mgd Head required: 95 ft Improved structure Wet well volume: 0.06 mg Switchyard Year: 1975	782,000	409,200
IV	Wastewater Pump Station Maximum flow: 25 mgd Head required: 75.2 ft Improved structure Wet well volume: 0.037 mg Switchyard Year: 1979	1,543,000	800,800

electrical costs are based on different cost functions for pump stations of this size. The estimated total construction cost for Case II is less than 5 percent lower than the actual cost.

4-5. No detailed cost breakdown was available for Cases II, III, or IV pump stations since these projects were bid lump sum. The project engineer provided the following general information about pump stations III and IV, which may explain why the MAPS cost estimates are low:

a. Case III is a sewage pump station with variable speed pumps housed in a belowground structure. A chemical injection system and a remote monitoring system are included.

b. Case IV is a sewage pump station housed in an elaborate aboveground structure, which was achitecturally matched to a nearby church.

With no additional information or an itemized cost breakdown, it is difficult to further rationalize the large differences between actual and estimated costs.

4-6. To determine if the MAPS costs were low or if the actual costs were unusually high, actual pump cost data (presented later in this section) for other projects were compared with MAPS. The data indicate that the contractor costs are unusually high or contain items such as very lengthy inlet and discharge lines, or feeder transmission lines that are typically not included in pump station costs. (These costs may be estimated separately by MAPS in the force main module.) Figure 4-1 summarizes the comparison of actual and estimated costs for the four pump stations discussed here.

Additional Verification

4-7. Figure 4-2 gives a comparison of MAPS cost curves and costs obtained from sources other than those used to generate MAPS costs. An EPA study (Pound, Crites, and Griffes 1979) developed cost curves for pump stations for raw sewage, preapplication treatment effluent, and final distribution based on published data, surveys of existing systems, consultation with construction contractors, and hypothetical costs based

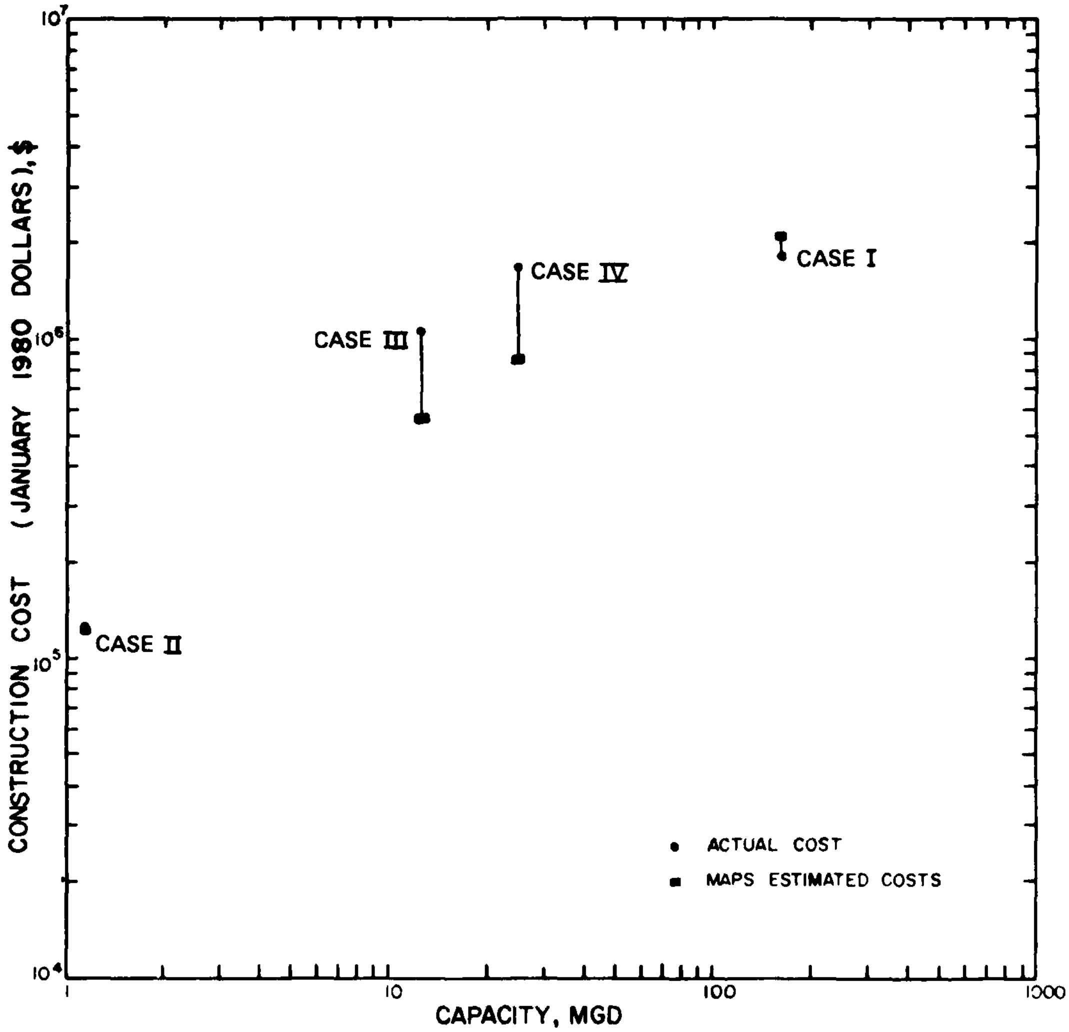

Figure 4-1. Summary of pump station cost estimates

on typical preliminary designs. These costs include a fully enclosed wet well/dry well type structure, pumping equipment with standby facilities, piping and valves within the structure, and controls and electrical work. Figure 4-2 gives these costs for a head of 300 ft. Costs are also presented for 100, 300, and 500 ft of head based on data gathered in the southern New England area (U. S. Army Engineer Division, New England 1977). These costs include the pump house, site work, instrumentation, inside piping, valves, auxiliary power generation, pumps, and standby pumps.

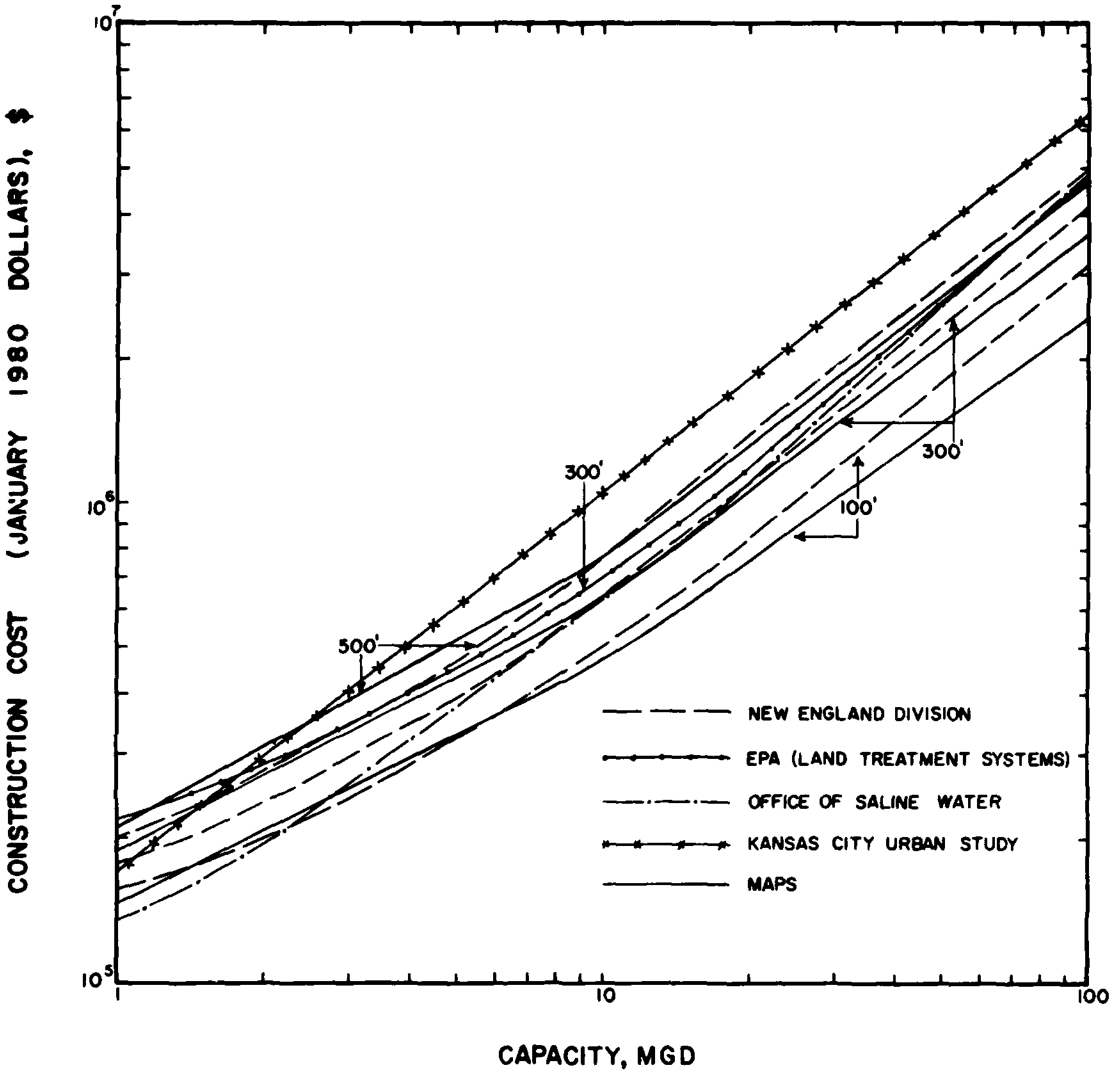

Figure 4-2. Comparison of functions for estimating pump station costs

4-8. The U. S. Army Engineer District, Kansas City (1978) sponsored a study in which cost curves were developed for pump stations as a function of flow only based on the maximum pumping capacity required during the useful life of the transmission mains. This represents a conservative estimate since pumps would, in practice, be added in stages as required by water demands. In a study by the Office of Saline Water (1966), treated water pump station costs are also given as a function of flow only. MAPS costs are presented for 100, 300, and 500 ft of head, based on an improved structure, wet well (volume dependent on flow), and switchyard. MAPS estimates show a good correlation with the available sources.

Program Modifications

4-9. A more thorough investigation of the actual pump station cost data from which MAPS cost functions were developed revealed that electrical and miscellaneous equipment costs are significantly affected by the number of pumping units per station. Therefore, the capability to input the number of units required was added to MAPS. The default values are based on maximum flow with a minimum of 2 to allow for a standby unit. In addition, miscellaneous equipment costs were expanded to include intake and discharge manifolds and valves, inlet and discharge lines within the structure, and the outlet structure as well as handling equipment and equipment for service facilities. It does not include intake structures such as those used in reservoirs, feeder transmission lines, or lengthy inlet and discharge lines.

4-10. Wet well costs have been updated based on data from Gumerman, Culp, and Hansen (1979). The cost is for belowground, reinforced concrete structures and includes instrumentation for control of the water level and for quality control operations.

4-11. Structure costs for small pump stations (less than 5.0 mgd) have been modified for increased accuracy in the 0.0- to 1.0-mgd range.

4-12. Mechanical costs have been adjusted to account for the effect of the type of station (water or wastewater). For wastewater pump stations, the mechanical cost is multiplied by 1.4, while for treated water, the factor is 1.0, and for small wastewater pump stations (<5.0 mgd), the factor is 1.2.

4-13. In conclusion, 30 percent of the sum of mechanical, electrical, structural, switchyard, and wet well costs is added to these costs to account for contingencies in obtaining the total construction costs.

Summary

4-14. Pump station construction costs may vary over a wide range depending on the type (low lift pumps, booster pumps, or high service pumps), the foundation treatment required, the level of architectural treatment desired, and the size and complexity of appurtenant features and control equipment. Items not included in MAPS cost functions, such as large reservoir intakes or lengthy inlet lines, must be identified and added to the costs determined by MAPS.

PART V: OPEN CHANNELS

Introduction

5-1. Costs and characteristics of the flow (velocity, Froude number) and the channel (earthwork volume) are calculated in the MAPS open channel module for lined trapezoidal channels. Channel dimensions and other design data may be input by the user or calculated by MAPS based on channel slope, flow, and Manning's n. Required input includes flow, length, canal invert elevations, and ground stations and elevations. Earthwork quantities and costs are calculated based on these data with consideration given to drop structures, which affect the profile of the channel.

5-2. Hydraulic data, including normal depth, velocity, Froude number, and wetted perimeter, are computed for up to four different channel flows. Costs are determined in the module for earthwork, canal lining, and various structures such as radial gates, siphons, irrigation gates, drop structures, bridges, and wasteways. Unit costs for numerous items, such as structural concrete and common and rock excavation, may be input by the user or determined by MAPS.

Input Data

5-3. The contractor provided design and construction cost data for a channel enlargement project as well as numerous hydraulic structures for other projects involving no actual channel excavation. In addition, detailed earthwork calculations were obtained from the Bureau of Reclamation for a section of the North Texas Canal, which was proposed as part of the West Texas-Eastern New Mexico Import Project.

Discussion of Results

Earthwork

5-4. A comparison of actual earthwork quantities with MAPS was not possible for the channel enlargement project due to the irregular cross sections of the original channel and lack of data at sufficient intervals to approximate trapezoidal cross sections. Therefore, only the Bureau of Reclamation data for the North Texas Canal was used to verify MAPS earthwork calculations. Table 5-1 summarizes the significant design data for the channel, as well as the actual and predicted earthwork quantities and costs. A comparison of totals for each reveals only about an 8-percent difference. Variances in the individual earthwork categories were expected due to slightly different definitions and design practices. For example, the quantity of spoil calculated by MAPS is higher than the actual quantity because MAPS assumes stripping for cross-sectional types other than "all cut" to be along the entire embankment. However, in this particular project, spoil volumes were computed based on stripping only along the compacted embankment. In addition, MAPS assumes stripping across the entire cross section, which in some cases falls in the excavated portion of the section. This also explains the fact that the volume of cut determined by MAPS is lower than the actual volume. The unit cost for stripping for this project was somewhat lower than the unit cost for common excavation. The costs calculated by MAPS were based on the same unit price for each.

Canal structures

5-5. Siphon costs are primarily a function of length and depth. Table 5-2 provides a description and cost comparison for 14 siphons constructed between 1973 and 1979 in Nebraska and Oregon. Overall, the average difference in actual and predicted costs is less than 11 percent. Based on this table, Figure 5-1 gives a graphical comparison of total costs. Irregular variations of cost with flow are due to the different siphon lengths and depths.

5-6. Radial gate costs are calculated as a function of gate area. The predicted costs for the nine gates listed in the tabulation on page 41 are within an average of 9 percent of the actual construction cost.

Table 5-1

Comparison of Channel Earthwork Quantities and Costs

Channel Description

Flow: 9000 cfs
Bottom width: 51 ft
Bottom slope: 0.000035
Side slope: 2:1
Normal depth: 25.5 ft
Unlined freeboard: 2.5 ft
Stripping: 3 ft
Length: 2.96 miles
Year: 1970

Type of Earthwork	Quantity, yd^3		Cost, $	
	Actual	MAPS	Actual	MAPS
Cut:				
Common	1,368,436	1,210,604	410,531	363,181
Rock	241,489	213,636	301,861	267,045
Fill	1,871,658	1,844,623	0	0
Borrow	1,067,566	1,267,973	311,270	380,392
Spoil	334,593	571,142	571,142	171,343
Total excavation	3,012,084	3,263,355	1,090,582	1,181,961

Table 5-2

Siphon Costs

	Siphons			Cost ($), ENR* = 2778	
Case	Flow cfs	Depth ft	Length ft	Actual	MAPS
I	11.95	10.3	514	26,029	22,294
II	7.96	6.2	1025	47,468	36,774
III	1.99	5.7	940	26,470	21,941
IV	6.98	8.2	494	19,380	17,674
V	5.98	25.5	765	23,478	30,341
VI	5.98	16.4	760	29,078	28,437
VII	5.98	7.5	1279	44,343	43,160
VIII	3.99	6.3	885	29,718	25,828
IX	4.97	9.6	623	19,724	20,539
X	7.96	6.2	422	16,825	15,134
XI	2.99	18.1	237	8,110	7,285
XII	5.98	16.6	903	30,946	33,836
XIII	1.99	8.7	657	17,838	16,216
XIV	238.70	68.8	3830	683,080	696,795

* Engineering News Record Construction Cost Index.

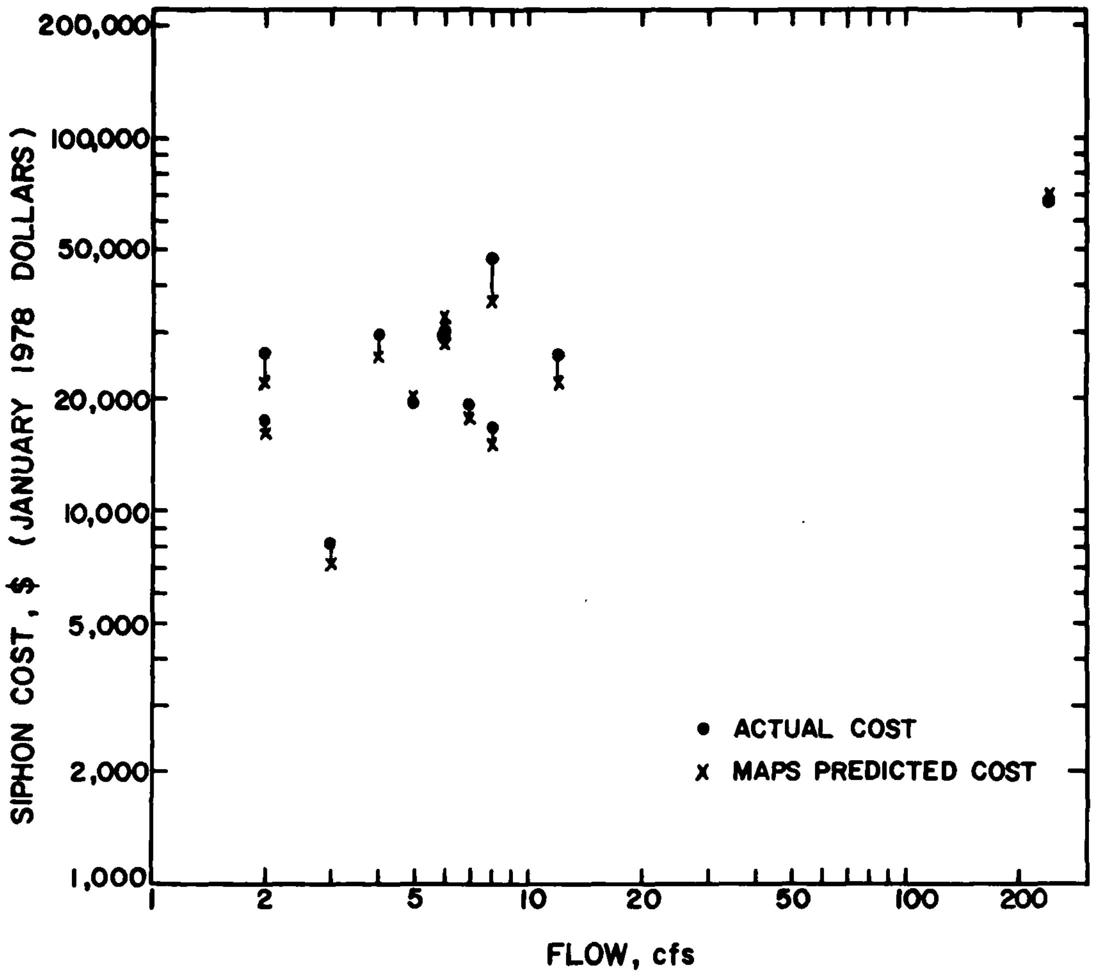

Figure 5-1. Comparison of siphon costs

Case	Area, ft^2	Construction Cost, $ (ENR = 2778) Actual	MAPS
I	153.0	21,000	19,700
II	137.5	19,000	18,500
III	167.8	21,000	20,900
IV	133.8	20,000	18,200
V	116.3	18,000	16,700
VI	133.3	18,000	18,200
VII	171.0	21,000	21,100
VIII	55.5	14,000	10,600
IX	42.1	12,000	9,000

Costs for smaller gates (less than 60 ft^2) are about 25 percent higher than the MAPS estimate. However, further studies (presented later in this chapter) confirm the accuracy of the MAPS costs. Figure 5-2 summarizes the comparison of actual and estimated costs for radial gates.

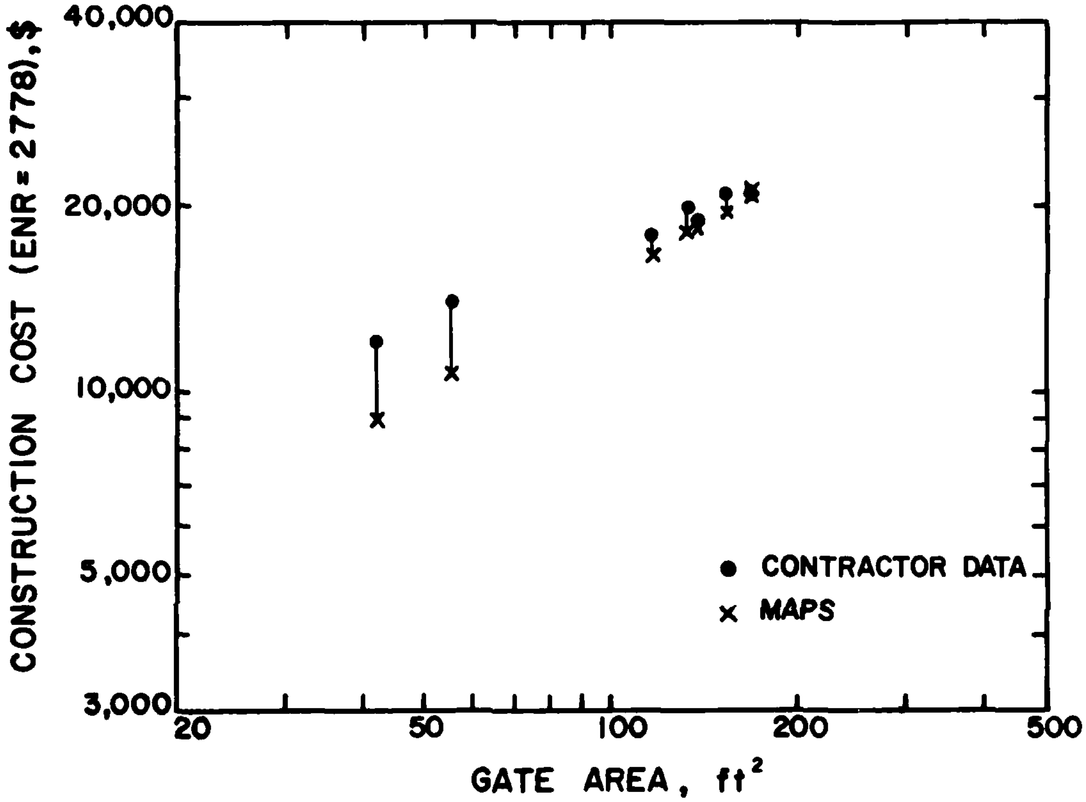

Figure 5-2. Comparison of radial gate costs

5-7. The contractor provided data for several different types of drop structures, including a vertical drop and check, baffle drop and check, simple vertical drop, simple baffle drop, and inclined drop. The following tabulation shows actual and estimated costs for various types and heights of drop structures.

Case	Flow, cfs	Height of Drop, ft	Construction Cost, $ (ENR = 2778) Actual	MAPS
I	671	6.15	64,000	55,200
II	671	10.20	63,000	56,300
III	671	3.70	56,000	54,300
IV	671	7.90	60,000	55,600
V	671	15.13	59,000	57,600

The differences in costs average only about 8 percent in spite of the variance in structure type. Cost functions for these structures are based on concrete volume, excavation, and weight of steel reinforcing. Actual unit prices for these components were used for the MAPS estimate.

5-8. Although the canal enlargement project included some additional structures such as wasteways, bridges, and irrigation gates, costs for these items were not given in sufficient detail to allow for a comparison with MAPS.

Additional Verification

5-9. Figure 5-3 gives siphon costs in dollars per foot as a function of flow and depth from surface to bottom. Actual data points from numerous Bureau of Reclamation studies are shown for comparison with MAPS cost curves. Depth was not specified for the Bureau's data, so an exact comparison cannot be made. However, all of the points are reasonably close to the MAPS estimates.

5-10. Figure 5-4 provides costs as a function of area for radial gates based on the MAPS cost function, actual data in the North Texas Canal Project, and data compiled by the Bureau of Reclamation from which a range of costs were determined. Again, MAPS is reasonably close to the other cost functions.

Program Modifications

5-11. The determination of normal depth in a trapezoidal channel

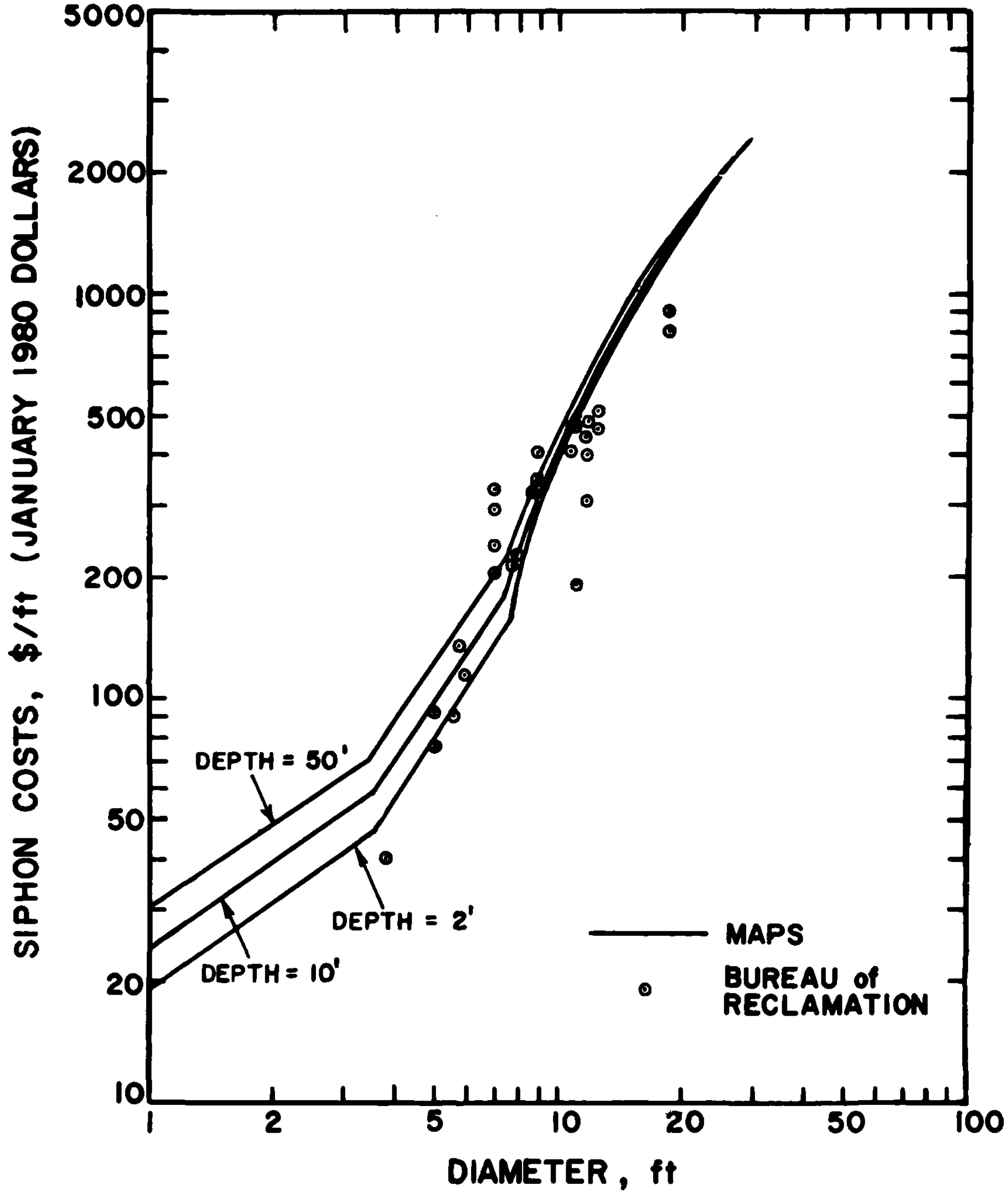

Figure 5-3. Additional siphon cost data

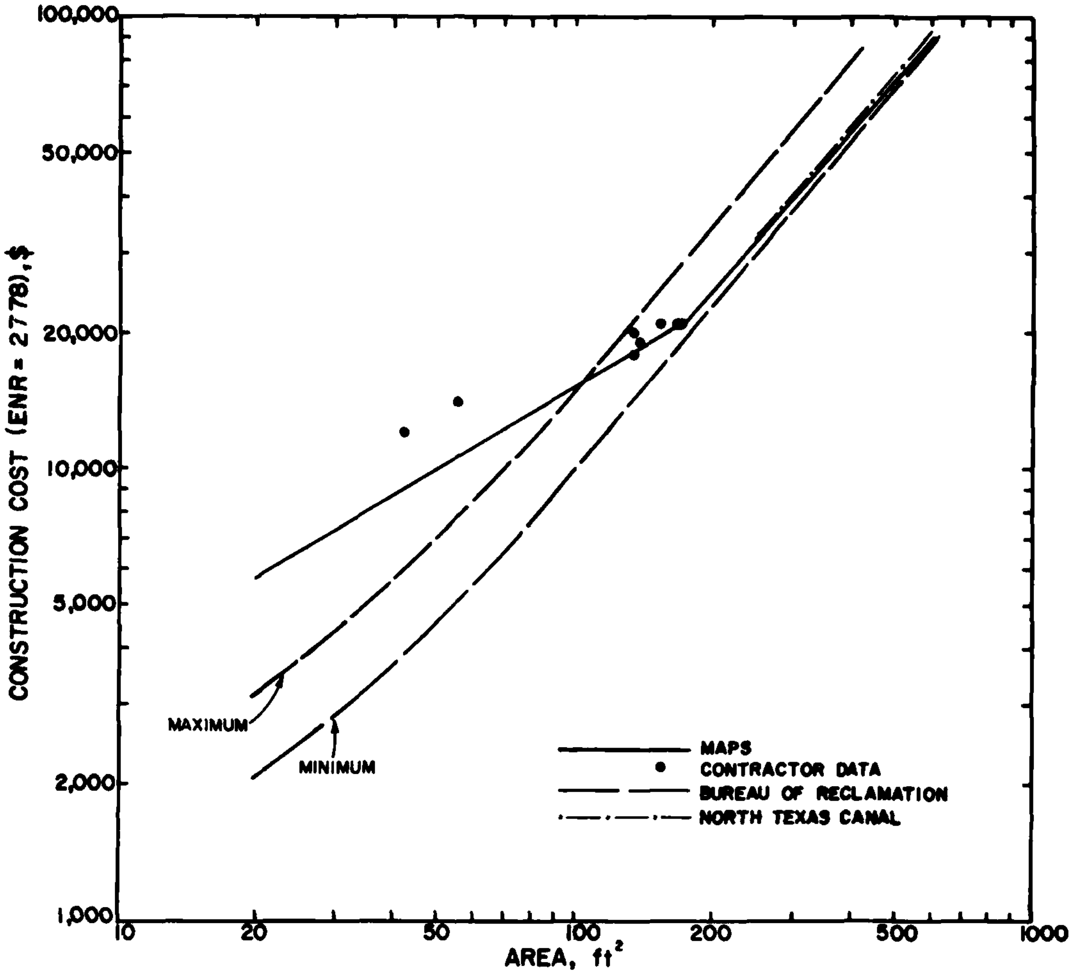

Figure 5-4. Additional radial gate cost data

involves a trial and error process. The method previously used in MAPS required a large number of iterations, depending on how close the initial guess was to the solution. In order to reduce computer time, the Newton-Raphson method has been incorporated in the MAPS procedure. The number of iterations required with this method depends on the percent accuracy (difference between predicted flow based on normal depth and actual flow) specified by the user or assumed by MAPS (accuracy = 1.0). In most cases, convergence occurs within 10 iterations, even when a high degree of accuracy is specified.

5-12. Slight modifications have been made in the calculation of siphon diameters for flows less than 850 cfs. Also, the cost of piping has been updated for smaller diameters (less than 7.5 ft). The cost of transition structures is included with the piping cost.

5-13. For increased accuracy, the cost function for radial gates has been separated into two parts, resulting in separate functions for small (less than 175 ft^2) and large (greater than 175 ft^2) gates. In addition, the unit costs used to develop the original function have been updated.

5-14. MAPS previously calculated the volume of concrete in drop structures based on actual data for structures in small channels (flow less than 80 cfs) with a narrow range of drop heights (4 to 10 ft). The updated function was based on drop heights of from 1 to 15 ft and flows up to 1000 cfs.

Summary

5-15. Open channel construction costs and quantities determined by MAPS show a very good correlation with data from actual projects. The calculation of earthwork quantities may be performed to a high degree of accuracy, keeping the following suggestions in mind:

a. The accuracy of the calculated quantities is only as good as the accuracy of the elevations input to the program. If more than the allowable 100 elevations are needed to accurately describe the terrain, the canal should be broken into two or more sections, with each section input separately.

b. The program will only accept input for five drop structures per run. The canal should be input in sections if more drop structures are required, with a maximum of five drops per section. Otherwise, a cumulative error will occur in earthwork quantities.

c. If head loss at a siphon is significant, the sections between siphons should be input separately. The initial elevation for each section after a siphon should reflect the head loss through the pipe. (Future plans include altering the program to account for the head loss automatically.)

5-16. Figures are presented in the MAPS documentation for five different channel cross-sectional types (all cut, all fill, etc.). These figures specifically show all assumptions made concerning stripping, cut, fill, etc., and should be referred to for exact definitions of quantities output by the program.

5-17. Estimated costs for canal structures investigated in this study also show a good correlation with actual costs. In general, the actual costs for structures will be slightly higher if the entire project consists only of providing the structures, rather than performing all the channel excavation in addition to the structures.

PART VI: STORAGE TANKS

Introduction

6-1. Storage tank costs are primarily a function of volume of storage and type of tank. However, other factors may significantly affect these costs and must be evaluated on an individual project basis. MAPS provides cost estimates for elevated steel tanks, ground level steel and prestressed concrete tanks, steel standpipes, buried concrete tanks, and excavated basins.

Input Data

6-2. Actual design and cost data were obtained for five storage tanks constructed since 1972. A description of each tank, the actual construction cost, and the construction cost predicted using MAPS are presented in Table 6-1 and Figure 6-1.

Discussion of Results

6-3. Actual costs for ground level concrete storage tanks are given in Cases I and IV. MAPS predicted construction cost for Case I is approximately 45 percent higher than the actual cost, while in Case IV the estimate is 13 percent lower. The project engineer for Case I provided additional information to rationalize the unusually low cost as follows:

- a. The storage tank was only a small part of the overall contract.
- b. Bidding for the project was highly competitive.

6-4. The MAPS estimated cost for elevated steel tanks is approximately 12 percent higher than Case II actual construction costs and 28 percent lower than Case III. Further investigation in Case III revealed an unusually sophisticated facility, justifying a higher cost than a "typical" facility of that size. The relative difference in

Table 6-1

Comparison of Actual and MAPS Costs for Storage Tanks

Case	Description	Construction Cost, $ (1980 Dollars) Actual	MAPS
I	Type: Concrete ground level Volume: 5 mg Year: 1974	400,000	580,000
II	Type: Elevated steel tank Volume: 0.1 mg Year: 1978	145,000	163,000
III	Type: Elevated steel tank Volume: 1 mg Year: 1975	717,000	516,000
IV	Type: Concrete ground level Volume: 8 mg Year: 1980	1,496,000	1,300,000
V	Type: Steel standpipe Volume: 0.75 mg Year: 1972	127,238	150,000

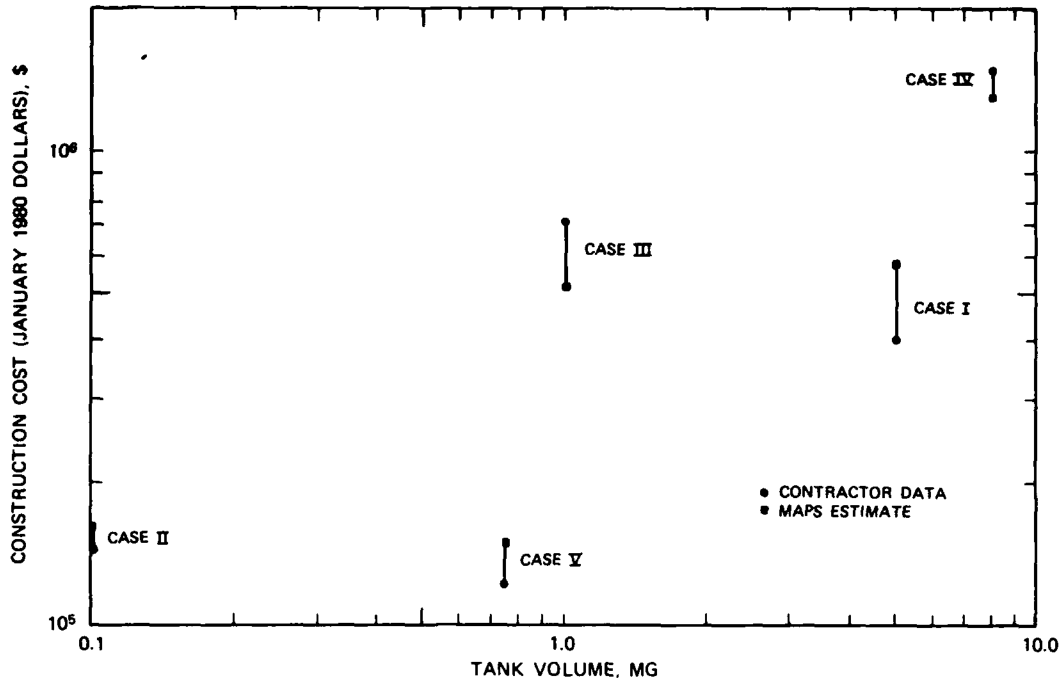

Figure 6-1. Summary of storage tank cost

construction costs for steel standpipes is less than 18 percent.

Additional Verification

6-5. Figures 6-2 and 6-3 give a comparison of MAPS cost curves and costs obtained from sources other than those used to generate MAPS costs for ground level and elevated steel tanks, and ground level concrete tanks. The relation of the case study cost data to the cost curves is also depicted. The Estimating Section of the National Park Service (the Denver Service Center) compiled storage tank costs based on data from previously constructed tanks in park areas (Borras et al. 1980). Dickson (1978) developed cost curves from water storage projects constructed in the southwestern United States. An Inter-University Task Force from Georgia (1979) developed costs for ground level steel tanks to be used for "ball-park" estimates in small public water supply systems planning. Figures 6-2 and 6-3 show that costs from these

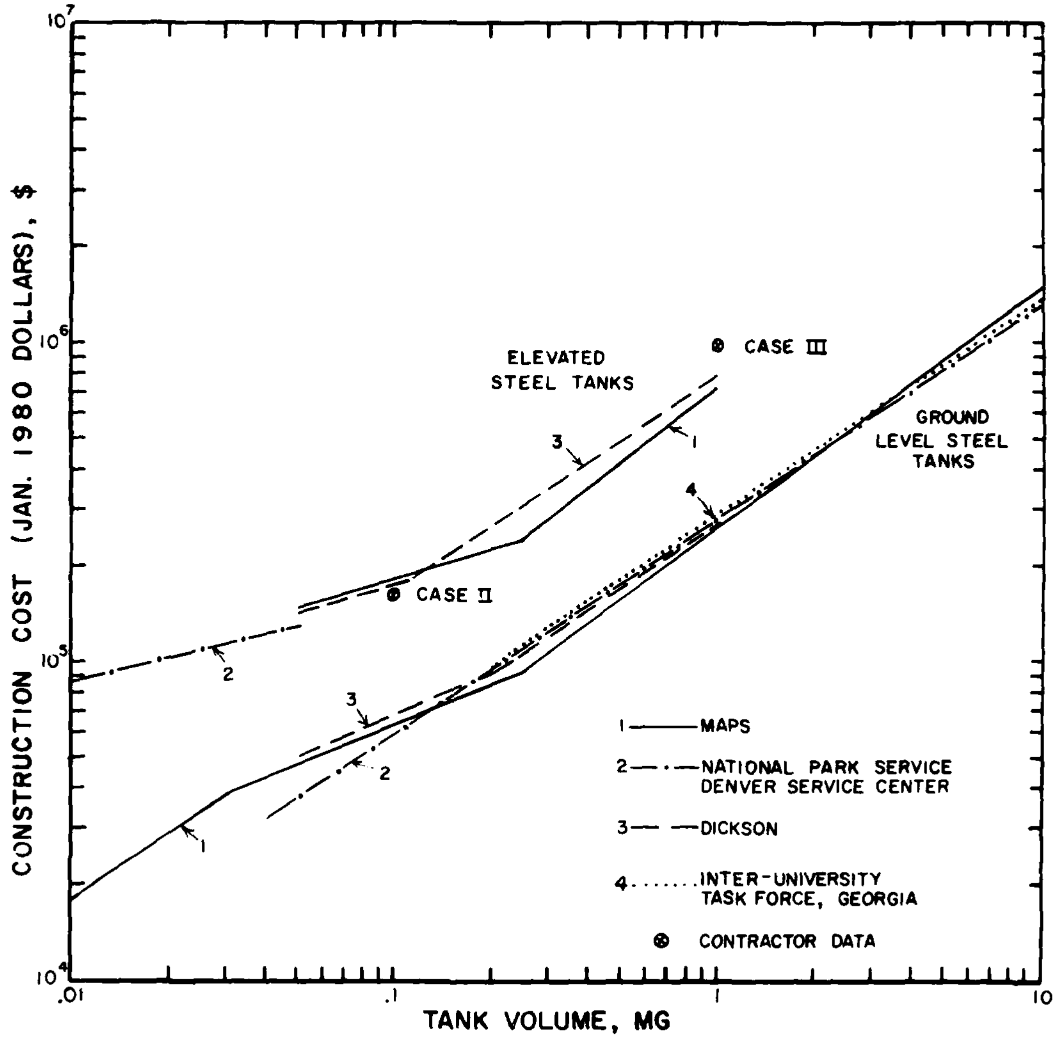

Figure 6-2. Construction costs for elevated and ground level steel storage tanks

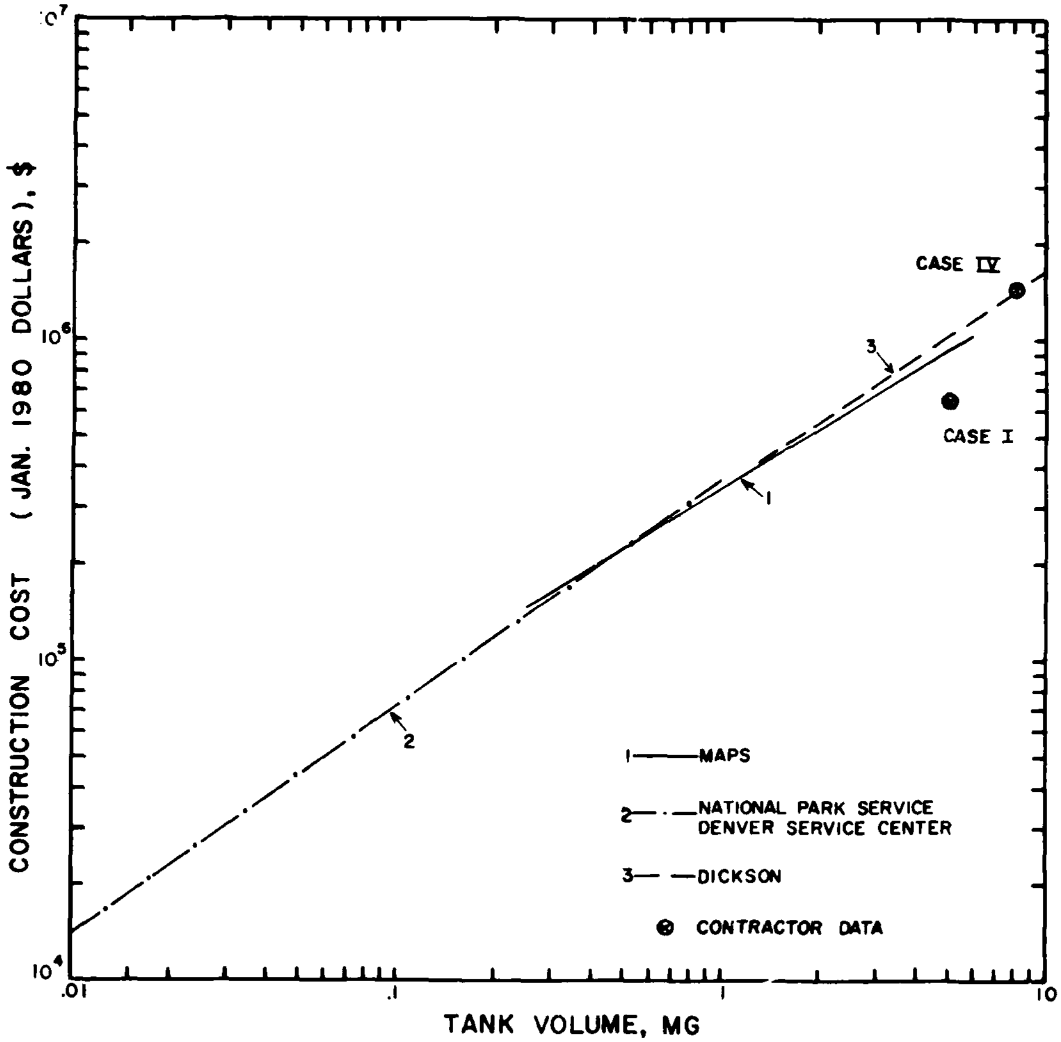

Figure 6-3. Construction costs for concrete ground level storage tanks

additional sources correlate closely with MAPS estimates even though they were developed from projects in different parts of the country for water systems in different types of areas (parks, small towns, and larger municipalities).

Program Modifications

6-6. Data from Dickson's (1978) study were utilized for the development of a cost curve for buried concrete tanks to be included in MAPS. The construction cost for these tanks is given in Figure 6-4 as a function of volume.

Summary

6-7. Accurate cost estimates for storage tanks may easily be obtained using MAPS, provided that certain factors are taken into consideration for each project. As previously noted, the size of the overall project significantly affects construction costs. When the storage tank cost is a small percentage of a larger total project cost, MAPS estimates may need to be reduced by as much as 25 or 30 percent. Other significant factors affecting costs are extensive foundation requirements for certain geographical areas and level of bid competition.

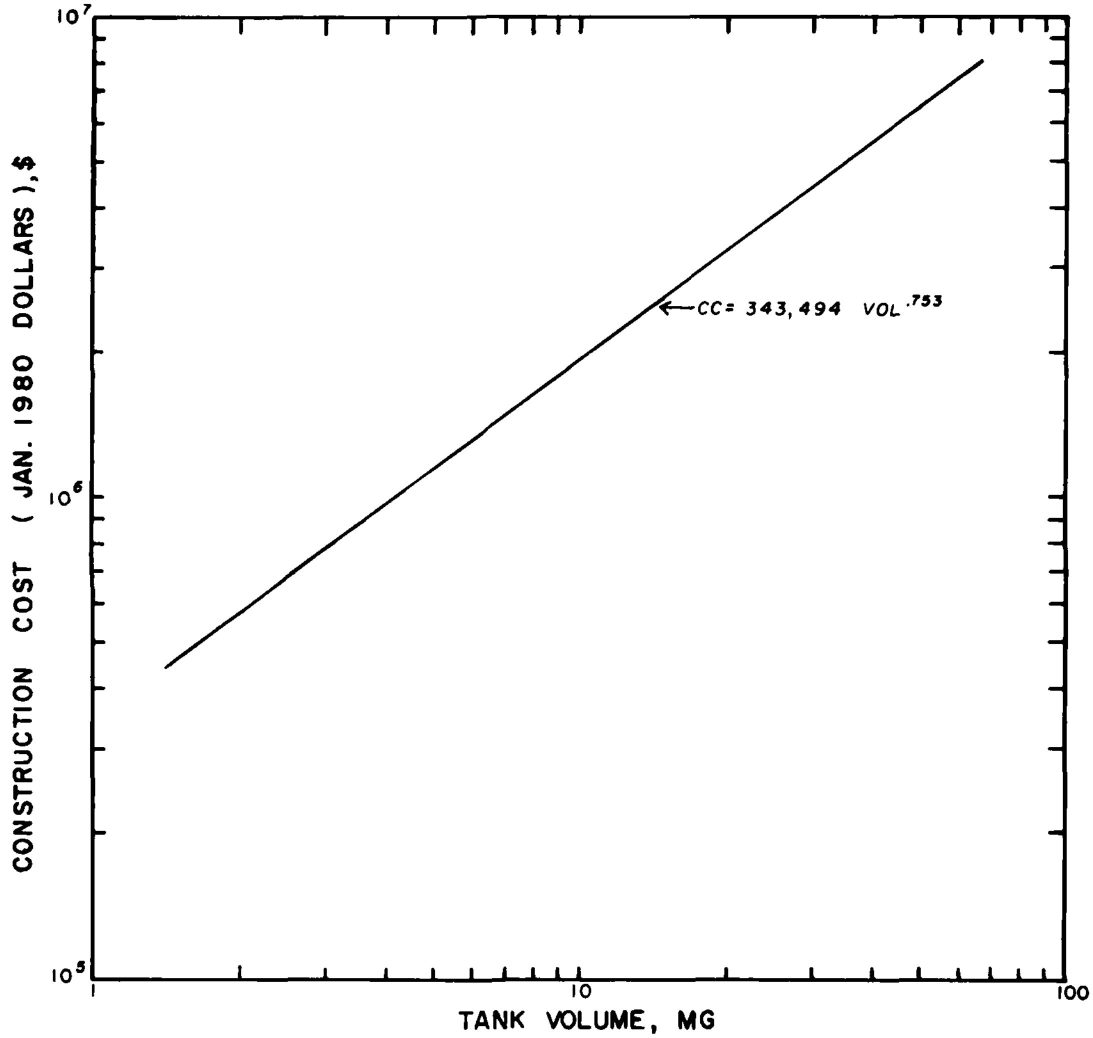

Figure 6-4. Construction costs for buried concrete storage tanks

PART VII: TUNNELS

Introduction

7-1. The MAPS tunnel module estimates construction costs for horseshoe-shaped drill and blast tunnels and circular machine-bored tunnels. Major elements in tunnel construction are as follows:

- a. Excavation. Excavation costs are based on the method used (drilling and blasting or machine-boring), tunnel width (specified by user or calculated based on flow), rock quality designation, and, for machine-bored tunnels, unconfined compressive strength.
- b. Control of water. Dewatering costs are based on a water inflow rate of 200 gal/min at the excavation face, with the water table 50 ft above the tunnel invert. These costs are calculated as a function of tunnel width, rock quality designation, and unconfined compressive strength. If dewatering is not specified, MAPS assumes none is required.
- c. Lining. Lining costs may be determined for watertight concrete linings. The cost is based on tunnel width and includes support and additional excavation beyond the finished tunnel diameter. If lining is not specified, MAPS assumes none is required.
- d. Inlet/outlet structure. Costs for inlet and outlet structures are based on square structures and a velocity of 2 ft/sec. The costs are calculated as a function of tunnel width.

7-2. MAPS estimates for tunnels are applicable for finished diameters greater than 10 ft and rock quality designations (RQD) greater than 40. Costs are not accurate for rock with an unconfined compressive strength greater than 40,000 psi. If the tunnel length differs significantly from 10,000 ft, the costs should be adjusted (costs per foot usually decrease slightly with increasing tunnel length). Also, if the project is small (less than $1 million), actual costs will be higher than the MAPS estimate because the percentage of total cost devoted to mobilization will be higher than for larger projects.

Input Data

7-3. Data were provided by the contractor for two tunnels constructed in Wisconsin in 1981. Table 7-1 provides a brief description of each tunnel, the actual construction cost, and MAPS predicted cost.

Discussion of Results

7-4. Costs for a lined, machine-bored tunnel are presented in Cases I and II. In Case I, the small tunnel diameter and low RQD index would indicate that MAPS cost functions are not completely applicable in this case (see assumptions discussed earlier in this chapter). The excavation cost was calculated for the minimum RQD index of 40, rather than the actual RQD of 15. Figure 7-1 gives excavation cost versus tunnel width for different RQD values (unconfined compressive strength assumed to be between 7,500 and 15,000 psi for this illustration). The figure shows that costs increase with decreasing RQD. Based on this, the MAPS estimate would be expected to be lower than the actual cost, as is confirmed in Table 7-1.

7-5. The tunnel diameter in Case II is also smaller than the minimum on which MAPS cost functions are based. In this case, however, the RQD index is well within the range for which the program is applicable. The estimated total cost is within 4 percent of the actual cost.

Additional Verification

7-6. Figure 7-2 gives costs per linear foot versus tunnel width as developed by the U. S. Bureau of Reclamation (1959). MAPS estimates and actual construction cost for the two tunnels discussed earlier are provided for reference. The Bureau's cost curve is a very general function, based on data from tunnels of a variety of shapes and lengths. Nevertheless, it does further confirm the accuracy of the MAPS estimates.

Program Modifications

7-7. Excavation and dewatering costs are a function of tunnel

Table 7-1

Comparison of Actual and MAPS Costs for Tunnels

Case	Description	Construction Cost, $	
		Actual	MAPS
I	Machine-bored RQD: 15 Compressive strength: 8,600 psi No dewatering Lined Length: 7,535 ft Width: 6 ft Year built: 1980	6,170,830	5,110,000
II	Machine-bored RQD: 60 Compressive strength: 10,000 psi No dewatering Lined Length: 7,535 ft Width: 5 ft Year built: 1980	3,688,625	3,817,809

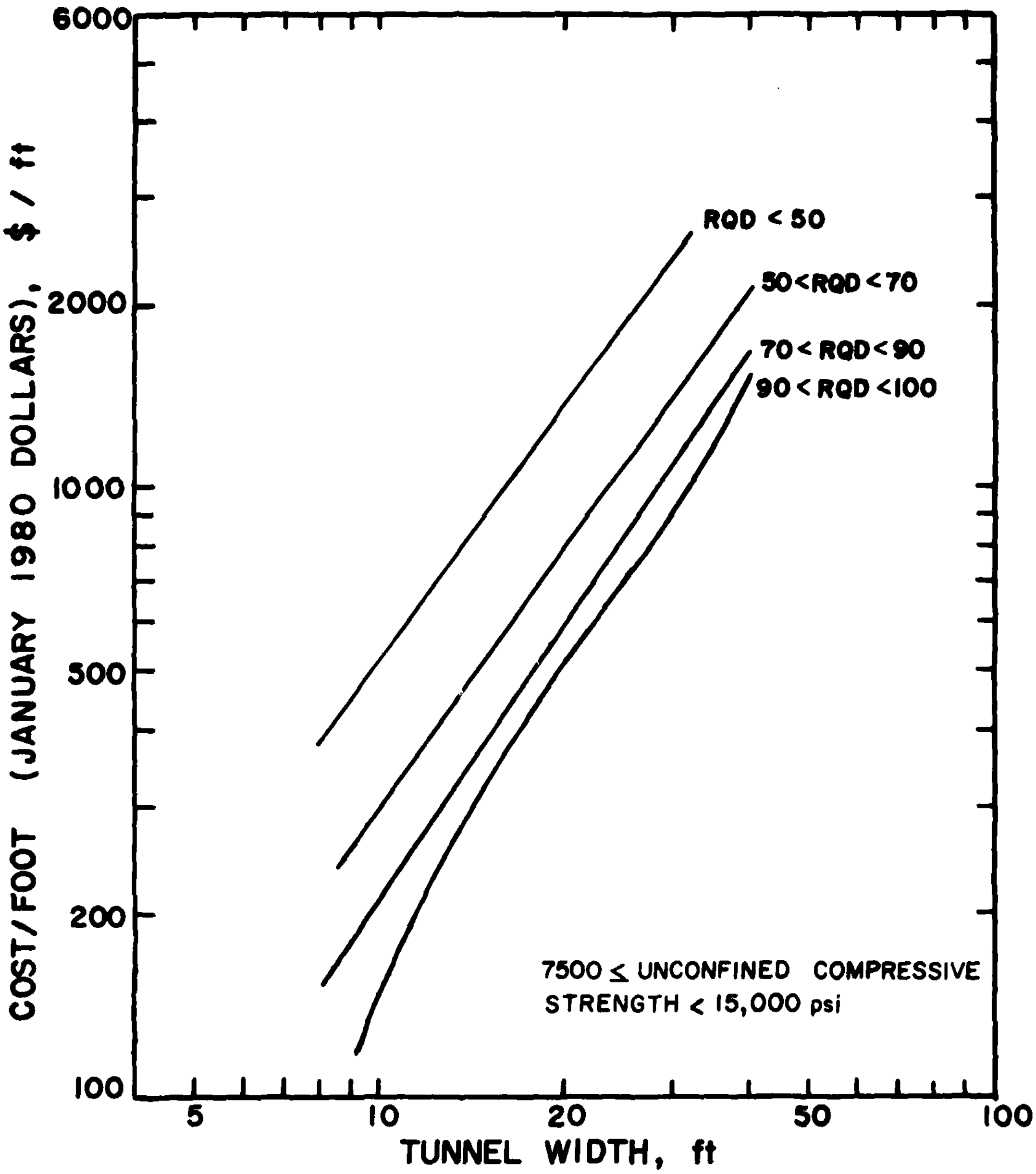

Figure 7-1. Unit costs of tunnels for different values of RQD

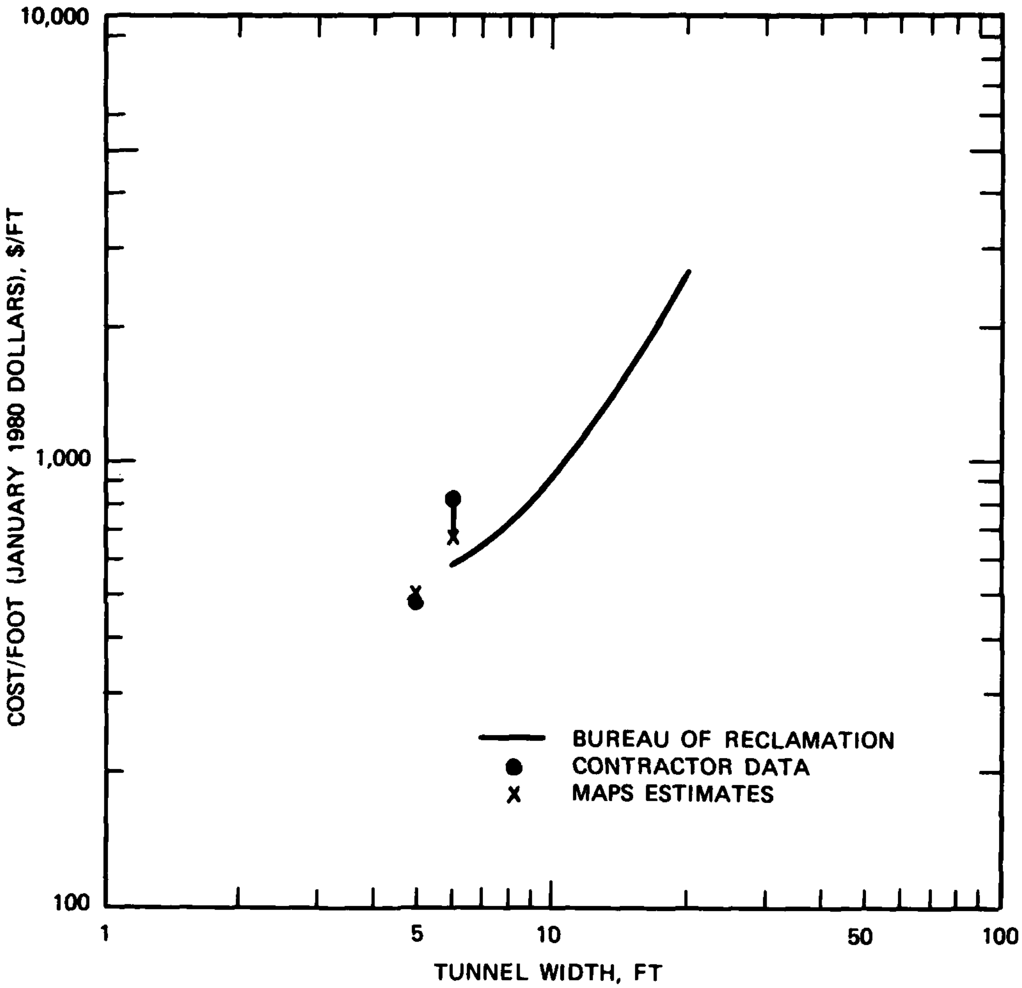

Figure 7-2. Comparison of MAPS and Bureau of Reclamation tunnel costs

width, RQD index, and unconfined compressive strength. A reevaluation of data from which MAPS functions were derived revealed a need to slightly modify the RQD and unconfined compressive strength limits for each function. The new limits are given in Appendix C.

Summary

7-8. Cost estimates developed for tunnel construction must be used cautiously, regardless of the method employed to obtain the estimates. Any subsurface geological condition undetected prior to construction may significantly affect the actual cost.

7-9. The results of this study indicate that MAPS estimates show reasonable accuracy even for small tunnel diameters, but are somewhat low for tunnels constructed in rock with an RQD index less than 40.

PART VIII: WATER TREATMENT PLANTS

Introduction

8-1. The MAPS water treatment module determines the cost of a treatment plant based on flow, unit processes used, and loading rates for the unit processes. The cost estimates are based on functions developed by Gumerman, Culp, and Hansen (1979) for EPA.

8-2. Verifying the MAPS cost estimates was a problem since the existing MAPS water treatment module is the result of only the first stage of a two-stage development process for the module. It contains most of the common unit processes for treating water, except lime softening, but does not contain costs for intake structures, sludge handling, in-plant pumping, clear wells, high service pumping, and administration buildings. The cost functions for some treatment processes (e.g., softening), sludge handling, in-plant pumping, and administration buildings are to be added during 1982. Cost functions for high service pumping and clear wells exist as part of the pump station and storage tank modules, respectively, and costs for intake structures are to be included in a separate module to be developed in 1982.

8-3. To verify the costs produced by the first stage of the water treatment module, it would be necessary to compare MAPS estimates for individual unit processes with costs provided by the contractor. Unfortunately, the contracts for the plants were let on a lump sum basis so costs could not be given for individual unit processes. Therefore, it was necessary to "hand calculate" costs using the Gumerman, Culp, and Hansen report for items not accounted for by the current version of MAPS and add these costs to the MAPS estimate to make the comparison with the actual plant costs provided by the contractor.

8-4. In paragraphs 8-12 through 8-14, the MAPS costs are compared with other cost functions without the correction for additional items. This is done to show the difference between MAPS and these cost estimates.

Input Data

8-5. Data were provided by the contractor for three conventional surface water treatment plants and two groundwater softening plants built in Florida, Alabama, and Colorado during the period 1972 through 1980. A more detailed description of each plant with actual and MAPS costs is given in Table 8-1.

Discussion of Results

8-6. Costs for the five treatment plants are shown graphically in Figure 8-1. Potential sources of error for each case are discussed in the following paragraphs.

8-7. Case I is a conventional surface water treatment plant for which the MAPS cost is low by 22 percent. This is primarily due to the fact that this 8-mgd plant is actually the first stage of a three-stage, 24-mgd plant. The yard piping and some other facilities have been sized for the ultimate flow of 24 mgd.

8-8. Case II is a softening plant for a groundwater source. The MAPS estimate is low by 18 percent. This plant contains two large (5 mg) buried concrete washwater tanks that were fairly expensive to build since the water table was very near the surface in this area.

8-9. Case III is a very large (125 mgd) surface water plant with conventional treatment. The MAPS cost was low by 12 percent. As in Case I, this may be due to the fact that this plant is the first stage of a larger (500 mgd), multi-stage plant. This plant is also fully automated and computer controlled.

8-10. Case IV is a conventional surface water plant. The MAPS estimate was high by 4 percent, probably because this project is an addition to an existing plant.

8-11. Case V is a small (5 mgd) softening plant. The MAPS estimate is high by 7 percent.

Table 8-1

Comparison of Actual and MAPS Costs for Water Treatment Plants

Case	Description	Construction Cost, $ Actual	MAPS

I	Flow: 8 mgd
	Year: 1979
	Intake and pumping
	Rapid mix and flocculation
	  Chemical feed: Alum
	  G Values: 600/sec, 50/sec
	  Detention times: 1 min, 32 min
	Clarification
	  Type: Rectangular
	  Depth: 12 ft
	  Loading rate: 540 gpd/ft^2
	Filtration
	  Type media: Dual
	  Loading rate: 2 gpm/ft^2
	Chlorination
	  Storage: Cylinder
	Sludge thickening: gravity
	High service pumping
		8,100,000	6,348,000

II	Flow: 30 mgd
	Year: 1972
	Groundwater treatment
	Softening
	  Raw water hardness: 300 mg/ℓ
	Recarbonation
	Filtration
	  Type media: Dual
	  Loading rate: 5 gpm/ft^2
	In-plant pumping
	In-plant storage
	  2 to 5 mg belowground concrete tanks
	Chlorination
	  Storage: Cylinder
	Sludge thickening: gravity
	Vacuum filter
	High service pumping
		6,640,000	5,428,000

III	Flow: 125 mgd
	Year: 1980
	Surface water treatment
	Rapid mix and flocculation
	  Chemical feed: Alum and polymer
	  Detention time: 1 min, 54 min
		50,000,000	43,856,000

(Continued)

Table 8-1 (Concluded)

Case	Description	Construction Cost, $ Actual	MAPS
III	Clarification Type: Rectangular Depth: 15 ft Loading rate: 3000 gpd/ft^2 Filtration Type media: Dual Loading rate: 5 gpm/ft^2 Chlorination Storage: Tank cars Ammonia Powdered carbon Clearwell 2 to 25 mg belowground concrete tanks Sludge drying beds 6- to 2-acre beds		
IV	Flow: 8 mgd Year: 1977 Surface water treatment Raw water pumping Rapid mix and flocculation Clarification Type: Rectangular Depth: 15 ft Loading rate: 560 gpd/ft^2 Filtration Type media: Dual Loading rate: 2 gpm/ft^2 Chlorination Sludge thickening: gravity Drying beds High service pumping	4,370,000	4,525,000
V	Flow: 6 mgd Year: 1973 Groundwater treatment Softening Clarification Type: Circular Depth: 15 ft Loading rate: 2800 gpd/ft^2 Recarbonation Filtration Sand filter Loading rate: 3 gpm/ft^2 Chlorination Storage: Cylinder	1,060,000	1,138,000

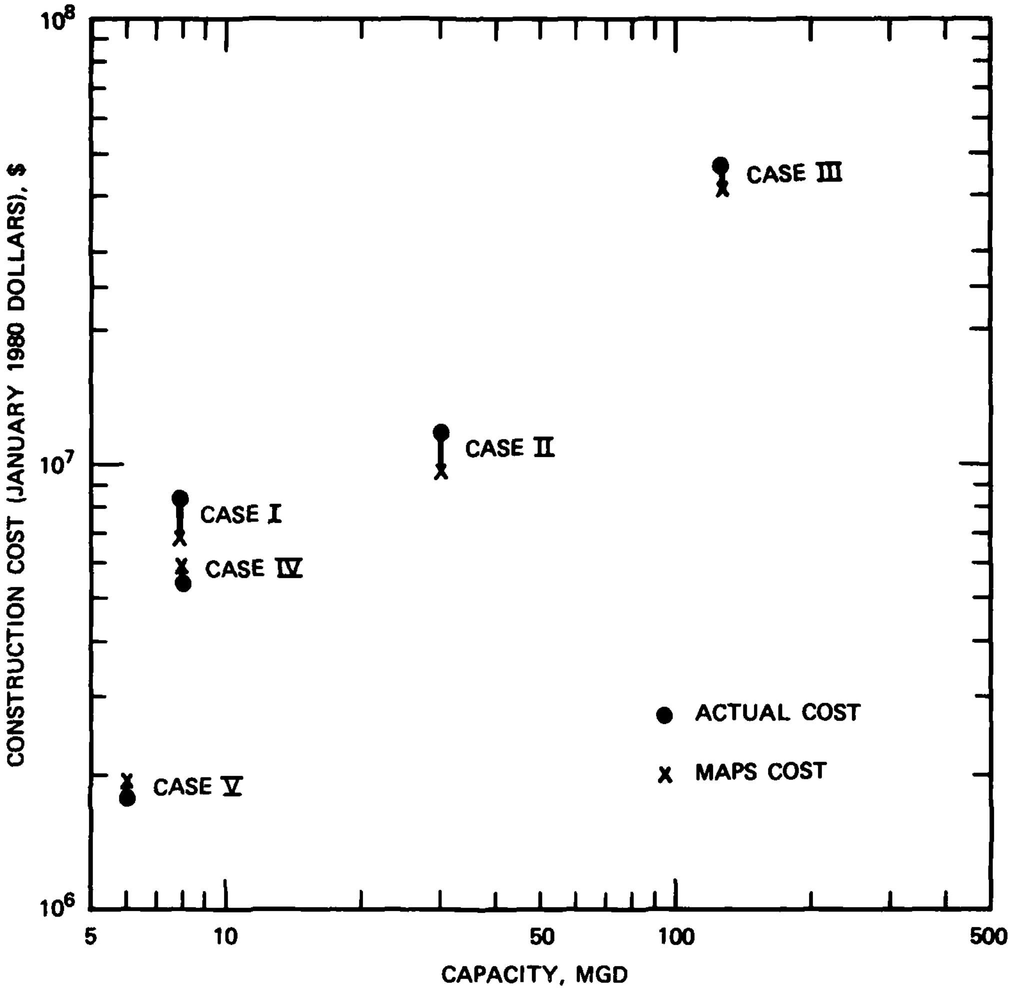

Figure 8-1. Comparison of actual and MAPS water treatment plant costs

Additional Verification

8-12. Costs functions are available for conventional treatment plants from a variety of sources. Some of the more recently developed functions are shown in Figure 8-2. The MAPS program was run for a conventional treatment plant (i.e. flocculation, clarification, filtration, and chlorination) for plants sized from 1 to 200 mgd. These costs are also shown in Figure 8-2.

8-13. The MAPS costs are lower because the current version of the

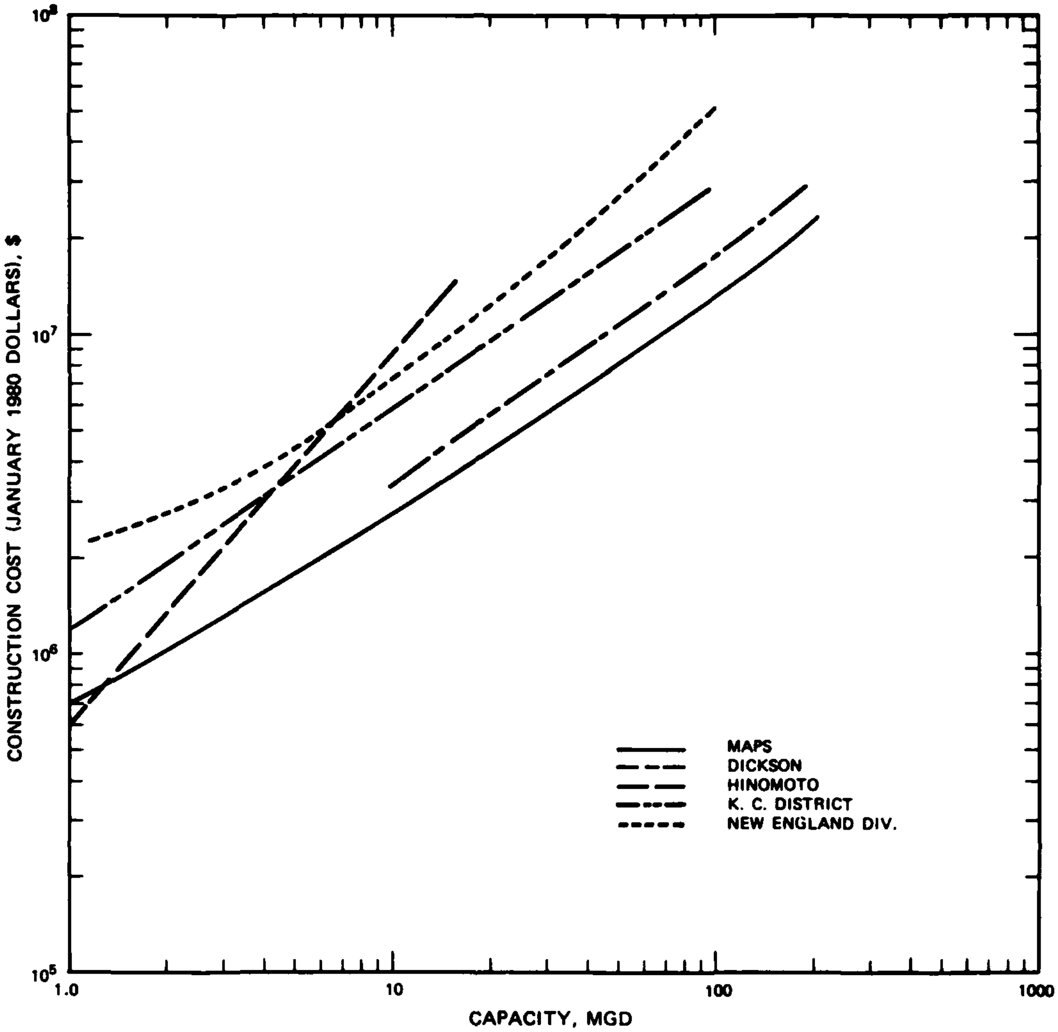

Figure 8-2. Comparison of cost functions for conventional treatment

MAPS water treatment module does not account for intakes, sludge handling, clearwells, high service pumping, and administration/laboratory buildings. When these cost items are included, the MAPS costs will be consistent with the other functions.

8-14. The comparison of costs for "conventional" treatment is somewhat imprecise in that for each study, "conventional" treatment is defined in a different manner. Dickson's (1978) function is based on a curve fit of 20 plants built on the west coast. Hinomoto's (1977) function is based on 12 plants with a range of flows from 2 to 20 mgd. The Kansas City District (1978) function was synthesized from a variety of sources for the Kansas City Urban Study. The New England Division (1977) developed its function based on actual bids on construction projects in New England. These costs do not include sludge handling or high-lift pumping and the function has a 95-percent confidence limit of $\pm$\$3.2 million.

Program Modifications

8-15. A major modification of the MAPS water treatment module is presently under way. After this modification, the module will include softening, sludge handling, in-plant pumping, high service pumping, and administration buildings. In addition, there will be a new MAPS module for intake structures.

8-16. Since the MAPS modification is currently under way, the revised documentation and user guide is not included in Appendix C, but will be published in another MAPS document.

Summary

8-17. The present MAPS water treatment module gives accurate costs for the unit processes it contains, although other costs must be added to give complete treatment plant costs. These other cost items are being added to MAPS. When this is completed, the verification exercise performed in this study will be repeated for the new module.

PART IX: WELLFIELDS

Introduction

9-1. MAPS wellfield cost functions are based on the assumption that all of the wells in the wellfield draw from the same aquifer, are drilled through the same material to the same depth, and have their flows collected and brought to one point for treatment and transmission. The five major elements in wellfield construction costs and the parameters on which each are dependent are as follows:

- a. Drilling cost. The drilling cost is based on the type of aquifer, the well depth, and the well diameter (a function of flow).
- b. Pumping cost. Pump cost depends on the type of pump (vertical turbine or submersible), the maximum flow, the head required at maximum flow, and the level of sophistication of the control equipment.
- c. Piping cost. Piping costs are based on the pipe length required to bring the flow to a single point and the pipe diameter, which is a function of flow.
- d. Housing cost. If housing is required, the cost is based on the maximum flow.
- e. Test well cost. If required, test well costs are a function of drilling cost and number of production wells.

Input Data

9-2. The contractor provided data for five wellfields constructed in Florida and Washington since 1975. Table 9-1 gives a list of the most significant design parameters for each wellfield, the actual construction cost, and the MAPS cost estimate.

Discussion of Results

9-3. Figure 9-1 gives a graphical comparison of actual total costs and MAPS predictions, based on the results presented in Table 9-1.

Table 9-1

Comparison of Actual and MAPS Costs for Wellfield Construction

Case	Description	Construction Cost Breakdown	Construction Cost, $ Actual	MAPS
I	Capacity: 10 mgd	Drilling	48,600	34,900
	Diameter: 14 in.	Pumping		
	Depth: 100 ft	Pumps and motors	86,000	64,800
	Number of wells: 10	Control equipment	86,100	72,200
	Shallow bedrock	Piping	225,000	265,000
	Vertical turbine pump	Test wells	15,600	5,500
	Test wells	Housing	216,900	192,000
	Improved structure			
	Year: 1975	Total	678,200	634,400
II	Capacity: 12 mgd	Drilling	520,000	183,700
	Diameter: 16 in.	Pumping	90,000	
	Depth: 1,400 ft	Pumps and motors		30,200
	Number of wells: 2	Control equipment		19,700
	Deep bedrock			
	Vertical turbine pump	Total	610,000	233,600
	No test wells			
	No housing			
	Year: 1975			
III	Capacity: 2.2 mgd	Drilling	73,447	14,300
	Diameter: 8 in.	Pumping		
	Depth: 70 ft	Pumps and motors	33,000	66,022
	Number of wells: 11	Control equipment	150,000	115,195
	Shallow bedrock	Piping	87,500	67,302
	Submersible pump			
	No test wells	Total	343,947	262,819
	No housing			
	Year: 1980			
IV	Capacity: 72 mgd	Drilling and test wells	261,000	207,600
	Diameter: 30 in.			
	Depth: 220 ft	Pumping	405,000	
	Number of wells: 5	Pumps and motors		404,400
	Unconsolidated	Control equipment		44,500
	Tubular wells	Piping	201,000	93,200
	Vertical turbine pump	Housing	539,000	331,900
	Test wells			
	Improved structure	Total	1,406,000	1,081,600
	Year: 1979			

(Continued)

Table 9-1 (Concluded)

Case	Description	Construction Cost Breakdown	Construction Cost, $ Actual	MAPS
V	Capacity: 14 mgd	Drilling	82,000	85,000
	Diameter: 20 in.	Pumping		
	Depth: 55 ft	Pumps and motors	35,000	59,000
	Number of wells: 5	Control equipment	170,000	48,000
	Unconsolidated	Piping	143,000	32,533
	Tubular wells	Housing	205,000	201,659
	Vertical turbine pump			
	No test wells	Total	635,000	426,192
	Improved structure			
	Year: 1980			

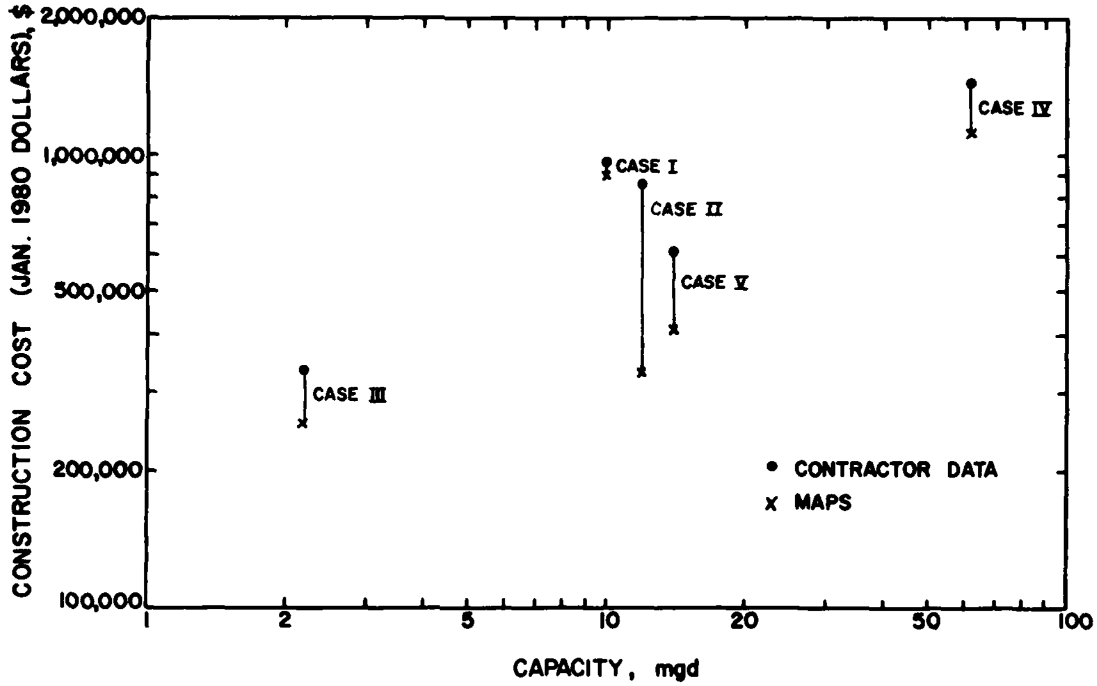

Figure 9-1. Comparison of wellfield construction costs

9-4. Costs for a wellfield consisting of 10 wells drilled to a depth of 100 ft in bedrock are presented in Case I. A cost comparison for each item shows a very good correlation, with less than a 7-percent difference between total construction costs.

9-5. The Case II wellfield costs include 2 wells drilled to a depth of 1400 ft in bedrock. The predicted cost generated by MAPS is considerably lower than the actual construction cost. The project engineer noted that this was an unusual situation, and, in further discussions, provided the following information:

a. The well was drilled through a highly productive, polluted aquifer, requiring special drilling equipment and techniques. These conditions also necessitated the use of heavy walled casing and an extensive amount of concrete.

b. Bids were accepted from only three contractors selected for their reputation of very high quality work, resulting in a closed market. This factor alone increased the cost to more than $100,000 above the engineer's estimate.

c. Specifications for the pumps were very strict. They were designed for total exterior operation with extra corrosion protection and a great deal of add-on equipment.

9-6. The predicted cost for the Case III wellfield, consisting of 11 wells drilled to a depth of 70 ft in bedrock, is within 24 percent of the actual construction cost. The abnormally high drilling cost is a result of the unusual casing configuration used, as well as the fact that one additional well was drilled and later abandoned due to low yield.

9-7. Estimates for 5 wells drilled to a depth of 220 ft in an unconsolidated aquifer are presented in Case IV. Actual piping costs in this case are for steel pipe, while the estimated cost is based on ductile iron pipe. In addition, the total length of piping was sized for total wellfield flow rather than for progressively increased flow down the pipeline for a linear arrangement of wells as assumed by MAPS.

9-8. Costs for a wellfield consisting of 5 wells drilled to a depth of 55 ft in an unconsolidated aquifer are presented in Case V. Steel pipe was used in this case, also. In addition, 72-in.-diam pipe was used over the entire length of the pipeline for a total wellfield flow of only 14 mgd. Pipes are sized in MAPS to flow at approximately 5 ft/sec at peak flow, resulting in diameters ranging from 14 to 30 in. progressing along the pipeline in a linear well arrangement. Control equipment costs reflect a level of sophistication not commonly encountered and, therefore, not accounted for in MAPS.

Additional Verification

9-9. Figure 9-2 gives a comparison of MAPS total construction cost estimates and the results of a study sponsored by the Tulsa District (Engineering Enterprises, Inc. 1980). The values of the parameters on which the study was based and from which MAPS costs were generated are also given.

9-10. Although the results of several other studies on wellfield construction costs are available in the literature, very little specific

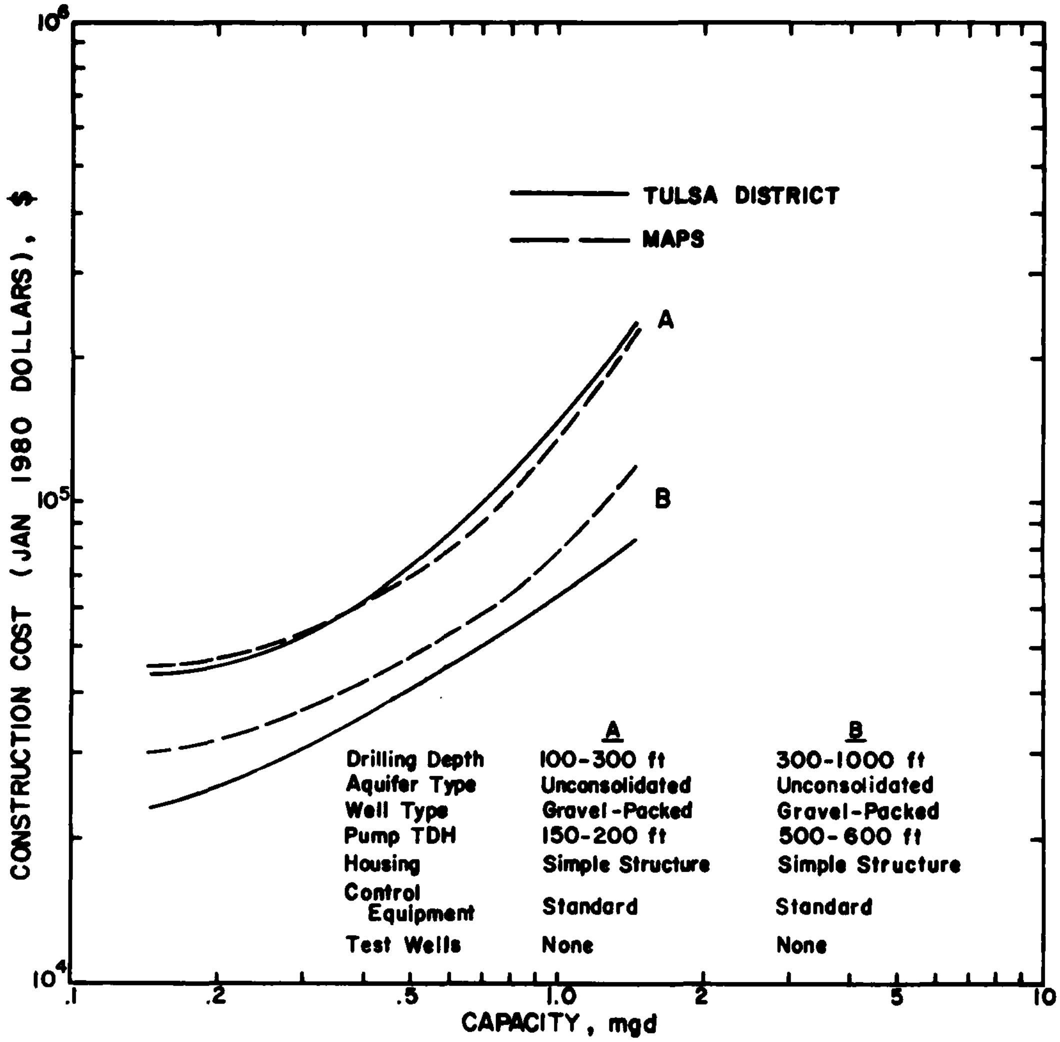

Figure 9-2. Comparison of MAPS unconsolidated, gravel-packed wells (TDH = Total Dynamic Head)

information is provided. For example, in a study sponsored by the New England Division (1977), one set of cost curves was developed as a function of capacity for two depths (75 and 150 ft) and aquifer transmissivities that result in individual well yields ranging from 175 to 2100 gpm. These curves are based on actual project data with a given range of well diameters and spacings and are intended for general comparative cost estimating. However, the validity of a comparison of MAPS estimates with these costs would be questionable without a further breakdown showing separate cost curves for each well diameter, aquifer type, etc.

Program Modifications

9-11. Cost functions for several elements of wellfield construction cost have been updated or expanded to provide additional options. Previously, when specifying an unconsolidated aquifer, costs were generated for gravel-packed wells. The user may now specify tubular (fully cased) wells or gravel-packed wells for this aquifer type by entering TUBULAR or GRAVEL PACKED in place of UNCONSOLIDATED. If no aquifer type is specified, MAPS assumes tubular wells in an unconsolidated aquifer. The diameter specified or determined by MAPS for tubular wells refers to the bottom of the bore hole, while the diameter of gravel-packed wells refers to the screen diameter (i.e. not including gravel pack annulus).

9-12. Housing costs have been modified to allow the user to specify a simple or improved structure by entering SIMPLE or IMPROVED in place of HOUSING. If HOUSING is entered, MAPS assumes a simple structure. If none of these options are specified, MAPS assumes no housing. Foundation costs are included in the housing cost.

9-13. When there is more than one well in the wellfield, MAPS has previously assumed that the wells were arranged in a circle, with the user specifying the wellfield radius by entering RADIUS XX.X MILES. Hence, the piping required to bring the flow to a center point was sized for flow from each individual well. The user now has the option of specifying an average distance between wells by entering DISTANCE XX.X MILES. The piping is then sized for cumulative flow progressing along the pipeline. For example, if three wells are specified with a capacity of 1 mgd each, the piping from the first to the second well is sized for 1 mgd, and the piping from the second to the third well is sized for 2 mgd. Figure 9-3 illustrates the difference between the two types of wellfield arrangements. In addition, pipe costs based on cast iron piping have been replaced with costs for more commonly used ductile iron piping with diameters ranging from 10 to 72 in. (polyvinyl chloride (PVC) piping is still assumed for diameters less than 10 in.). The unit cost of piping includes excavation and backfill (assuming a rectangular trench), a depth of cover of 5 ft, and open country terrain.

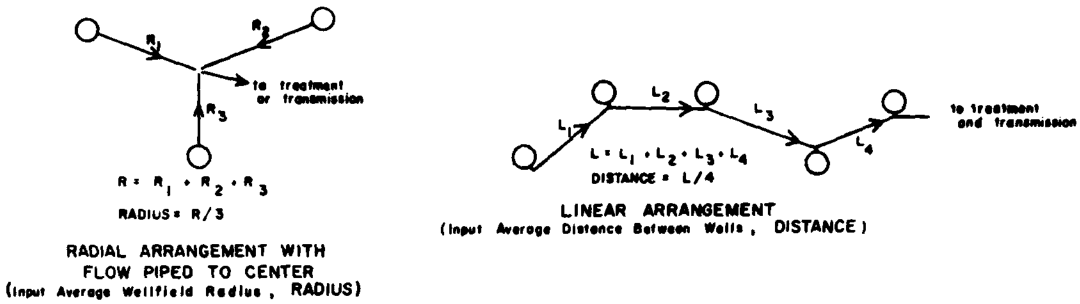

Figure 9-3. Definition sketch of wellfield arrangements

9-14. Pump control equipment may now be specified with the costs added to the basic pumping cost. Discussions with various manufacturers and engineers indicate that costs for control equipment are dependent on number of wells and level of sophistication of the equipment. The user may enter STANDARD CONTROLS to indicate a system that monitors the flow, pressure, and whether or not the pump is running. SOPHISTICATED CONTROLS may be entered to indicate a system with remote monitoring capabilities with costs for one remote station included. If NO CONTROLS is entered, or if neither of the other options are specified, MAPS assumes no control equipment other than what is already included in the pump cost. Although there is a tremendous variation in costs for these control systems, depending on the number and complexity of features included, the costs in MAPS provide good planning level estimates. However, if detailed information is available for a particular system, the control cost may be input by the user rather than calculated in MAPS by entering CONTROL SYSTEM COST OVERRIDE XX.X $.

Summary

9-15. Wellfield construction costs are very difficult to predict due to the extent of their dependence on geographical location, local market conditions, and geological environment. When applying MAPS estimates to a specific situation, familiarity with local conditions concerning drillers and labor should be reflected in the CITY adjustment factor. If geological conditions do not approximate any of the options provided in MAPS, drilling costs may also require an adjustment. For example, well costs in igneous and metamorphic rocks may be considerably greater since the cost per foot of drilling could be significantly higher. However, this should seldom represent a problem since the overwhelming majority of water wells are drilled in sedimentary rock, to which MAPS costs apply.

9-16. Other factors may vary from MAPS assumptions, including: (a) number of test wells required (dependent on probable success ratios in drilling for a specific area); (b) length of pumping tests; (c) type of drilling equipment used (cable tool, rotary, or other); (d) whether or not test wells are converted to production wells; and (e) level of sophistication of pump control equipment. However, total well construction costs are not highly sensitive to these types of variables.

9-17. The comparison of MAPS estimates with the Tulsa District study (Engineering Enterprises, Inc. 1980) and the actual construction costs from the contractor reveals an excellent correlation, other than in situations deemed out of the ordinary by the project engineers.

PART X: SUMMARY AND CONCLUSIONS

10-1. Comparisons were made between actual costs and MAPS estimates for 35 different facilities. (Total excavation, siphon, radial gate, and drop structure costs were treated as individual facilities for open channels.) The results of these comparisons are shown in Figure 10-1. All of the points would fall on the 45-deg line in Figure 10-1 if the program were perfectly accurate. The points are all fairly close

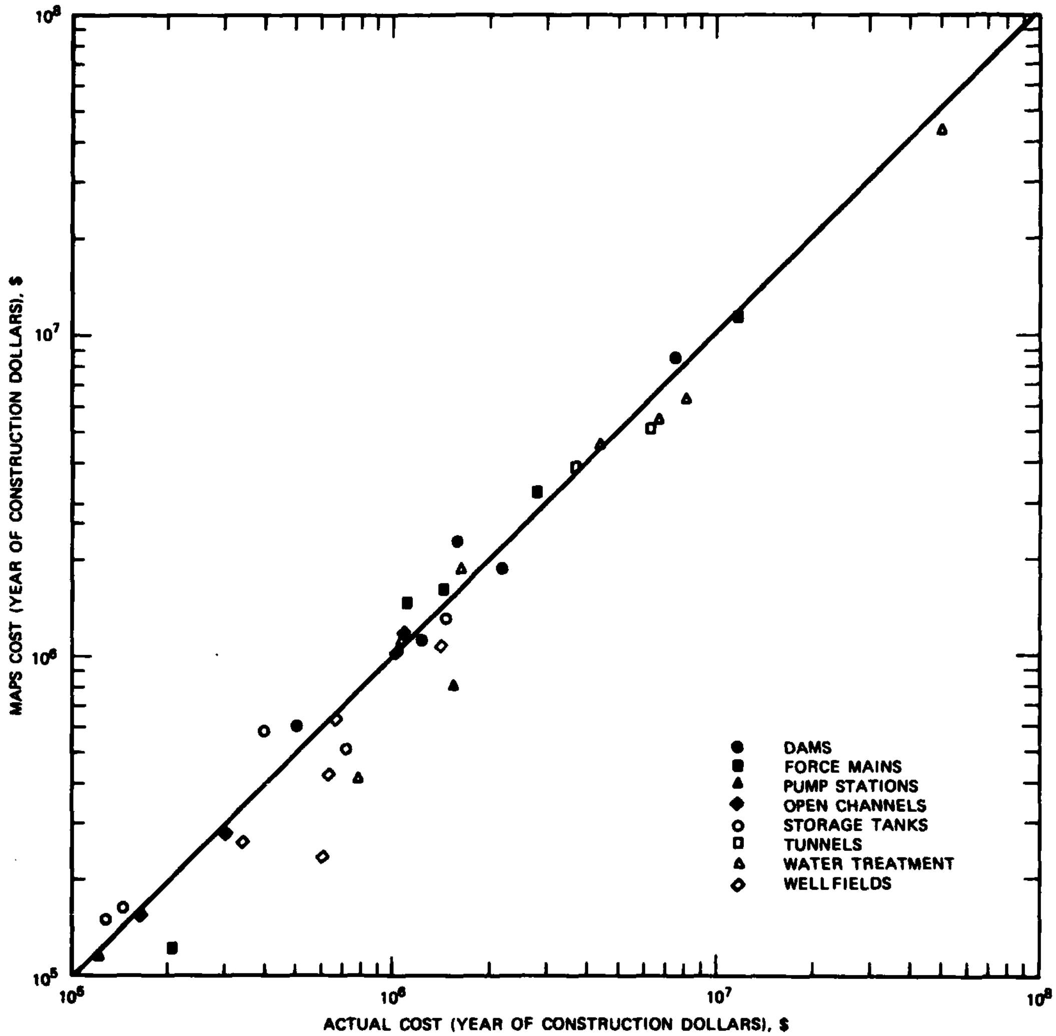

Figure 10-1. Summary of actual and MAPS costs

to the line, indicating a high level of correlation between the actual and MAPS costs.

10-2. The geometric mean percent error for the 35 facilities is 13.9 percent. Existing guidance in ER's and EM's on cost estimating indicates that estimates are considered accurate if they fall within 25 percent of actual costs. The MAPS costs estimates for 75 percent of the facilities fall within this 25-percent range. The cost estimate for only one facility differs by more than 50 percent.

10-3. Comparison of the MAPS cost functions with other cost functions shows that the MAPS costs are consistent with other sources of cost data. This indicates that the facilities for which MAPS was not very accurate are unusual cases which MAPS is not designed to handle. Such facilities could conceivably be identified beforehand and considered separately. Nevertheless, MAPS is considerably better than generalized cost functions in accounting for the many variables affecting costs, as generalized cost functions usually have only one or two independent variables.

10-4. As the result of this study, many of the shortcomings of the program (e.g. limited range of cost functions and difficulty in accounting for some important variables) were identified and corrected. In the case of water treatment plants, the program is being upgraded independently of this study.

10-5. In general, the MAPS computer program produces cost estimates that are of acceptable accuracy for planning studies.

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APPENDIX A: MAPS--A PLANNING TOOL FOR CORPS OF ENGINEERS REGIONAL WATER SUPPLY STUDIES*

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MAPS – A PLANNING TOOL FOR CORPS OF ENGINEERS REGIONAL WATER SUPPLY STUDIES[1]

Thomas M. Walski[2]

ABSTRACT: A computer program (MAPS – Methodology for Areawide Planning Studies) has been developed by the U.S. Army Corps of Engineers, Waterways Experiment Station to assist planners in producing a comprehensive array of alternatives without sacrificing the detail and accuracy of the analyses. MAPS is a set of computer based models which can be used to simulate the water resource alternatives and to develop planning level design and cost estimates. Two application examples are discussed. The Salinas-Monterey (California) Urban Study sought to identify and determine cost of combinations of water source, transmission, and treatment to meet an array of water needs in future years. The Nashville (Tennessee) Urban Study had similar objectives but the output was prepared on a service area basis for more than 40 such units. Using MAPS it was possible to prepare planning level design and cost estimates for a very large number of alternatives.
(KEY TERMS: water supply; water distribution; cost estimation; planning.)

The U.S. Army Corps of Engineers has been assuming a larger role in planning assistance in water supply recently through the Urban Water Resources Studies Program and somespecially authorized planning studies. Unlike flood control and navigation studies which many times lead to construction of projects, the Corps role in water supply is one of providing local decisionmakers with technical assistance in the form of an array of feasible alternative plans at an "intermediate" level of detail.

The array of alternatives developed by the Corps planners must be comprehensive, yet sufficient attention must be paid to the details of costs and benefits to make the plans realistic and accurate. Add to the above requirements, the fact that these regional studies encompass large geographical areas, and one can appreciate the problems facing the planner trying to produce a study that is sufficiently comprehensive, yet detailed, within limited time and manpower.

DESCRIPTION OF MAPS PROGRAM

To assist planners in producing a comprehensive array of alternatives without sacrificing detail or incurring large costs, the Environmental Laboratory at the U.S. Army Engineer, Waterways Experiment Station (WES) has developed the MAPS (Methodology for Areawide Planning Studies) computer program (U.S. Army Engineers, Office, Chief of Engineers, 1979). MAPS is a set of computer-based models which perform water balance calculations and develop planning level design and cost estimates. It can be used to identify problems and measures, and then select the least cost facilities to make up these measures. By computerizing the cost, design, and flow balance computations, MAPS can save the planner a great deal of work while allowing him to investigate a very large number of alternatives.

MAPS has been written for use by planners who may or may not have a background in computer programming. The only equipment required to run the program is a small computer terminal and a telephone. The input and output is interactive and keyword oriented so the user can have instant results. All data entered are stored in the data base from run-to-run so that the user need only enter data which he wishes to change for that run.

MAPS was conceived so that the Corps could have consistent, easily updatable, centralized systems which would save each District the trouble of performing the ground work (e.g., collecting cost data) for such a system for each study. MAPS can also be used with the CAPDET (Computer Assisted Procedure for Design and Evaluation of Alternative Wastewater Treatment Systems) program which was also developed at WES, for waste water management studies.

Unlike most computer programs which are developed to solve a single problem or type of problem, MAPS is a multipurpose program with each "module" capable of being run independently of the others while having the capability of data transfer with other modules. As a planning tool, MAPS has more in common with a carpenter's tool box rather than a hammer or a saw.

The scope of MAPS can best be shown by describing the functions of the various modules in the program.

Water Balance

The water balance (also called simulation) portion of the MAPS allows users to simulate the system behavior for a variety of flows and water use and management schemes. The system

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is represented by a network of links and nodes. Nodes are places where something happens to the water (e.g., service areas, treatment plants, diversions) while links are the conduits connecting these nodes (e.g., streams, pipes, canals).

Flow enters the network through headwater nodes or along stream reaches in the case of naturally occurring waterbodies, or from wellfields or pump stations for water which is removed from ground water storage or transferred from outside the basin. Flows can be simulated under steady state conditions for any season of the year or under unsteady conditions through droughts of various lengths.

The planner is usually interested in the impact of man's activities on the flows at given points in the network. Water uses are represented by service area nodes which contain water use data as a function of time period (e.g., 1980, 2000, 2020); use sector (e.g., municipal, industrial, agricultural) consumption and loss (e.g., percent return flow); and time of year (e.g., April, September). Up to four of these population and water use projections can be stored in the program for each service area.

The planner instructs the program as to the conditions (e.g., reservoir operation, time period) he wishes to simulate and the program responds with the flows at designated links and nodes. This allows the user to (1) identify water source problems, (2) select capacities of treatment and transmission facilities in the design modules, and (3) assess the adequacy of existing and proposed reservoirs under the given hydrologic and water use conditions. The MAPS data base system makes it easy to perform sensitivity analysis as only the parameters changed from one run to the next need be typed to rerun the program.

Design Modules

The design and cost modules in MAPS produce planning level cost estimates and design parameters, given the limited description of the facility that the planner usually has during a study. The MAPS costing procedures can produce cost estimates which are usually within the accuracy of detailed cost estimates at only a fraction of the cost.

The costs are generally more accurate than generalized cost curves available in the literature since the MAPS costs account for many of the independent variables impacting on costs while cost curves are a function of one or two variables. For example the pipe costs in MAPS are not only a function of diameter and length of pipe but of the depth of excavation, quantity of rock, number of values, hydrants, bends, type of pipe, and type of terrain to name a few of the variables which can be accounted for by the program. If the user does not wish to specify these detailed variables, he need only specify flow, and initial and final elevations, as the program will use "default" values for the parameters to be specified (e.g., depth of cover = 3 feet, laid across open country). These default values permit the user to concentrate on the parameters believed significant to the study.

The costs for each module are divided into construction cost of each item, total construction cost, overhead cost, land costs, total capital cost, amortized capital cost, operation and maintenance labor cost, material and supply cost, power cost, total operation and maintenance costs, and average annual cost. All costs are calculated by the program except for the "land cost" which is used to account for site specific items such as land purchase, easements, special site preparation and relocations. Economic data are shared by all of the design routines. This includes construction cost indices, interest rate, power cost, O&M wage rate and base year.

The design modules in MAPS are described below.

Force Main. This module calculates the cost and head required for pipes which are flowing full. The data required to run the routine is the peak flow and length. (The initial and final elevation are needed if the required head is to be calculated.)

The required head is calculated using Bernoulli's equation. The friction losses are calculated using the Darcy-Weisbach Equation and a numerical approximation to the Moody diagram. The roughness factor is a function of the type of pipe specified. Roughness factors and costs are available in MAPS for asbestos cement, cast iron, ductile iron, PVC, prestressed cylinder, pretensioned cylinder, reinforced concrete, and steel pipes. Minor losses are calculated if bends or values are specified.

The hydraulic output includes velocity, velocity head, friction loss, minor head loss, and head required for up to nine different pipe diameters which may be specified by the user or selected from standard sizes by the program to produce reasonable velocities. The calculations are done for up to four flows corresponding to peak and average flow (Stage 1) and peak and average flow (Stage 2). This staging information may be used to size pumping stations. If sufficient elevation head is available to drive the flow, the actual flow through each diameter pipe is also calculated.

The construction costs are divided into pipe and other costs. Pipe includes the costs of the material and laying the pipe. These costs depend on diameter and type of pipe. Other costs include excavation, values, hydrants, lands, and contingencies. These costs depend on depth of excavation, amount of rock excavation and side slope of trench. A summary of capital, O&M, and average annual cost is also printed.

Gravity Main. This module gives costs for circular pipes which are designed not to flow full. The minimum data required to use the program are the flow and change in elevation.

The slope and Manning's N is used to calculate the required pipe diameter. Depth at minimum flow is calculated based on the hydraulic elements chart to check the velocity at that flow so that scour velocities are adequate. Drop manholes can be specified along the pipe to reduce slope.

The construction costs are based on length, diameter, depth of excavation, amount of rock excavation, type of land use, and a local multiplier. Operation and maintenance costs are based on the average flow.

Open Channel. The open channel design module calculates costs for trapezoidal or semicircular channels based on the slope and the flows. The program uses Manning's equation to determine the normal depth and cross-sectional area of the channel.

The costs are based on either earth or concrete lined channels. Siphons and bridge relocations can be included in the costs calculated by MAPS as well as the amount and cost of cut and fill required. The land costs and costs for other relocations must be calculated outside of the program.

Pipeline. The pipeline design module is a combination of the force main and pump station modules. The pipeline module runs the force main and pump station design calculations simultaneously to minimize the pipeline cost.

The module iteratively selects different diameter pipes and calculates head losses and pipe costs as in the force main module. The output from the module is used to select the number of pump stations and size, stage, and cost of these stations. The equivalent annual cost of the pipeline is calculated for the entire set of facilities for different diameter pipes.

The input is the same as the force main and pump station input. The output is essentially the same although the user also gets a summary of the annual cost of each component in a summary table. The user can also suppress some of the longer output tables.

Pump Stations. The pump station module calculates the construction, operation, and maintenance costs of a pump station (or series of pump stations). The flow and head required from the pump station can be input by the user or passed to the pump station routine directly from the force main design routine.

The user can also specify the type of structure, whether a wet well or intake is desired, if several or a single station is desired on a pipeline and the percent of time the pumps are operating. The construction of the pump station can be staged. In this case, the user specifies the year in which the second stage is to be constructed. All of the structure will be built in stage one while electrical and mechanical equipment will be added at the second stage.

The cost output divides the construction costs into mechanical, electrical, structural, intake, wet well, and miscellaneous costs. The operation and maintenance costs are divided into power labor and materials.

Reservoir. The reservoir routine calculates reservoir costs as a function of area required, inflow, valley shape, and height of dam.

The cost items are divided into embankment, power plant (if desired), spillway, intake and outlet, waterway, clearing, and land. The cost of land and relocations is especially important in this module since this can amount to an extremely large portion of the costs. The costs are also corrected for the region of the country.

Storage Tanks. The costs of storage tanks are calculated as a function of tank volume and type of tank. Cost data exists in the program for excavated basins, ground level tanks, steel standpipes, and elevated water tanks.

The excavated basins can be lined with bentonite, asphaltic material, PVC, or butyl neoprene. Embankment protection can also be specified for the excavated basins.

Tunnels. The tunnel design module calculates the costs for large tunnels (> 10 ft. wide). The costs depend on the length, width, compressive strength of rock, rock quality designation, type of construction, lining material, and water control.

The cost of drill and blast tunnels are based on horseshoe shaped tunnels while machine bored tunnels are based on circular shapes. The size of the tunnel can be input by the user or calculated by the program based on flow, length, and change in elevation.

Water Treatment. The water treatment cost module calculates the costs of a water treatment plant as a function of the unit processes used and flow. The module can be used to determine the cost of a single plant, compare costs of alternative unit processes to treat a flow or determine the effects of various design parameters on flow.

MAPS views the treatment plant as a series of up to ten "blocks" which can contain alternative unit processes. The module calculates the cost for each possible combination of processes (train) in the blocks, and ranks these trains according to average annual cost.

The output consists of the construction, O&M, and average annual cost. These costs are summed to give total O&M cost while overhead and land costs are added to the sum of the unit process construction costs plus interfacing. The average annual cost is calculated from the capital and total O&M cost.

It is also possible to list the design parameters used for each unit process (e.g., loading rates, detention times). The user can control the level of detail of the output.

Waste Water Treatment. MAPS does not contain a waste water treatment design module. MAPS users wishing to perform preliminary design and cost calculations for waste water treatment plants are referred to the CAPDET computer program mentioned earlier. It provides more detailed design and cost information than MAPS but is easy to use.

Wellfield. The wellfield design module determines the cost of drilling wells and installing pumping equipment and piping if more than one well is used. The costs are based on drilling depth, depth to ground water, drawdown, and flow.

The user can stage the construction of wells within the wellfield and specify shelter for the well, various types of material to be drilled, distance between wells, efficiency of pumping equipment or whether test wells are required.

The construction costs are divided into drilling, pumping, testing, housing, and piping. The operation and maintenance cost are divided into power and labor.

Hardy-Cross Network Analysis

In addition to the design and cost modules, MAPS contains a Hardy-Cross Network solution routine to determine the head at nodes and the flow in pipes for a looped or tree structured pipe system. The user need only identify the elevation of each node and the diameter and length of each pipe to describe the system to the program.

Then given elevation at some tank(s) and/or pressure at some pump(s) the water use at some node(s), the program balances the flow in the network. The Hazen-Williams equation is used to calculate head losses in each pipe.

The user can control the accuracy of the solution by overriding the default convergence criteria for the program. The solution is efficient in that repeat runs of a network for different flow rates use the previous solution as a starting point for the iterative solution.

Amortization

The MAPS amortization module calculates the present worth and average annual cost of future sums, equal payments between two years and linearly varying series of payments between two years. The module can produce tables which summarize all of the payments for a project and give the overall present worth and average annual cost.

SALINAS-MONTEREY URBAN STUDY

The first application of MAPS was to Water Supply task of the Salinas-Monterey Urban Study in early 1978. The study area consisted mainly of Monterey and Santa Cruz counties along the Pacific Coast of central California (U.S. Army Engineers, San Francisco District, 1975). The purposes of the study were to (1) formulate and evaluate intermediate level water supply plans for the decisionmakers, and (2) test the MAPS program in a real-world planning study.

There were two distinct subareas in the study area: (1) Santa Cruz County where the problems consisted chiefly of providing municipal and industrial water to populated areas from an integrated system of surface and ground water sources, and (2) Monterey County (especially the Salinas Valley) where the problems consisted of managing the ground water supply to support the demands of irrigated agriculture. In Santa Cruz County, MAPS simulations were used to identify measures to meet future water needs and the design modules were used to determine the costs. In the Salinas Valley, the water source problems were investigated by the U.S. Geological Survey using a finite element model and MAPS was used only to determine the costs of the measures.

The study was coordinated by the San Francisco District, Corps of Engineers and corresponded to what is called a "Stage 2" study (Development of Intermediate Plans). The final Stage 2 report (U.S. Army Engineers, San Francisco District, 1978) was prepared by a consultant to the District using input from the MAPS study done at WES (Walski and Gibson, 1979) and the U.S. Geological Survey ground water model study (U.S. Geological Survey, 1978).

In Santa Cruz County reservoir sites had been identified in an earlier study (Creegan and D'Angelo-McCandles, 1968). Information about these sites and stream flows for the design dry cycles were used to simulate the water supply alternatives for three sets of population and water use projections. The study was conducted at the end of the drought that had hit California in the mid 1970's. This period was used as the critical dry cycle for the study.

The simulations indicated that there was not enough rain falling on the study area during dry periods to meet future needs. The solution to water supply problems would require integrated development of ground water supplies supplemented by reservoir construction and/or importation of water from the California Water Project.

A very large number of possible arrays of alternatives were identified using the simulations. The possible combinations were presented to the decisionmakers in staging diagrams which gave the year in which the facility was required for a given plan as a function of the population and water use projection.

Once the staging of construction for the alternative plans was developed, the required facilities and their capacities were identified and designed using the design modules. Eighteen pipeline projects and seven new or upgraded water treatment plants were designed and costed. (At this time the MAPS pipeline routine was not prepared so the pipelines were designed by piecing together force mains and pumping stations.)

These designs were pieced together with the reservoir costs, which were already available, to arrive at the cost of each alternative under the base population projection. The average annual cost of each facility was also presented as a function of year constructed to allow local decisionmakers to assess the costs of delaying or advancing construction.

In Monterey County, the study consultant identified a number of water supply measures based on the results of the USGS ground water model. The MAPS design routines were used to cost the facilities that comprised these measures. These facilities included wellfields, pipelines, and canals with siphons.

NASHVILLE URBAN STUDY

The water supply portion of the Nashville Urban Study was also a Stage 2 Study (U.S. Army Engineers, Nashville District, 1976). Unlike the Salinas-Monterey study area where water was scarce, there is a great deal of water in the Nashville Study area from the Cumberland River which runs through the center of this ten-county area in Middle Tennessee.

The problem in the study area is the rapid growth of smaller towns in the hills away from the Cumberland River. The question is whether it is cheaper to pump water from the Cumberland or develop ground water or reservoir sources away from the Cumberland.

The Nashville District hired a consultant to prepare the water supply report (U.S. Army Engineers, Nashville District, 1979) with input from the USGS (U.S. Geological Survey, 1979) who conducted a ground water study and the MAPS analysis of alternatives conducted at WES (Corey, *et al.*, 1978).

MAPS was used to generate stream flows with 50-year recurrence intervals, and these flows were simulated through the year 2030. Very few small surface water sources were adequate for future needs. The most promising surface water sources were the Cumberland River and the Columbia Dam proposed by TVA. The USGS identified adequate ground water in many places to meet the study area needs.

The problem then became one of determining the costs of developing the sources and transmitting and treating the water. The MAPS pipeline optimization routine was developed to help select optimum pipe size and quickly determine costs.

From two to five alternative facility plans were prepared for each of the utility districts in the study area. The amortization calculation also gave the costs of buying and selling water between the utility districts. The output was presented by utility districts unlike the Salinas-Monterey Urban Study where it was up to the local decisionmakers to allocate the costs.

SUMMARY

While water supply planning is primarily a local responsibility, considerable savings can be realized by using regional water supply systems. Planners studying these regional systems are faced with a large number of possible alternatives from which he must identify feasible solutions and then non-inferior solutions to present to decision makers.

The MAPS program was developed at WES to relieve the *planner of a considerable amount of the* computational burden which allows him to concentrate his efforts on issues which cannot be addressed using the computer. Using MAPS the planner can develop accurate information as to the sources of water and the costs for constructing and operating the required facilities over time.

The MAPS program has been shown to be a viable planning tool in two Corps Urban Studies. Using MAPS a much larger array of alternatives could be addressed than by conventional means without sacrificing accuracy of the cost estimates.

ACKNOWLEDGMENTS

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APPENDIX B: WATER RESOURCES PLANNING
COST ESTIMATING TOOLS*

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WATER RESOURCES PLANNING COST ESTIMATING TOOLS

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Abstract. Water resources planners often need to determine costs of proposed projects yet do not have time and money to perform detailed designs. This paper presents a discussion of the alternative methods for producing these estimates, and describes two computer programs developed by the U. S. Army Corps of Engineers, which overcome many of the shortcomings of the other approaches.

Keywords. Water resources; computer-aided design; urban systems; water pollution; cost estimating; sensitivity analysis; economics.

INTRODUCTION

Water resources planners are often faced with the problem of developing cost estimates for sets of alternatives. The costs for these alternatives are functions of a large number of variables that must be taken into account for the cost estimates to be accurate. The planner also does not wish to spend a great deal of time and money to develop these estimates. He is, therefore, torn between detailed cost analyses based on preliminary engineering designs, and crude approximations based on a usually limited set of past observations.

This was the situation encountered by the U. S. Army Corps of Engineers in the early days of their Urban Studies Program (i.e., water resources studies for urban areas). The Corps identified the need for a consistent, inexpensive, easy-to-use, updatable approach that could provide cost estimates that could account for large numbers of possible variables faced in real-world planning problems. This article describes the selection of the approach used, including a discussion of the advantages and disadvantages of different approaches, and illustrates the use of the methods developed by the Corps.

AVAILABLE APPROACHES

Several approaches to planning level cost estimating were identified. These can be divided into three overall categories: 1) cost element, 2) historical data, and 3) parametric costing. The features of these approaches are described below.

Cost Elements

In the cost element approach, the quantity of every item (i.e., cost element) is determined (e.g., cubic yards of concrete, number of valves) and the cost is multiplied by the unit cost of the item, which may also be a function of the quantity required. While this approach is used in construction estimates and provides very accurate costs, it is usually inappropriate for planning level cost estimates. First, it takes a great deal of time and effort to perform a design of sufficient detail to arrive at the quantities required for the estimate. Second, it is usually difficult to determine the unit prices (especially in larger sizes) for items used in water resources studies. For example, standard cost estimating references such as the Dodge Guide (McMahon, 1978) and Means Cost Data (Godfrey, 1979) do not contain data for radial gates or pressure reducing valves, much less contain these costs as a function of size. Third, quantities of labor and materials for operation and maintenance are very difficult to obtain.

The planner, using the strict cost element approach, is faced with the decision of spending a great deal of time and money developing the estimates or restricting his investigation to two or three alternatives. Restricting the analysis in this manner has the effect of eliminating some possibly viable alternatives *a priori*. In Corps Urban Studies, it was important to investigate a comprehensive array of alternatives. Therefore, the strict cost element approach was judged unacceptable for planning level cost estimating.

Historic Data

If a project has recently been built, is currently operating, and is identical to the alternative under consideration, the capital and operation and maintenance cost of this project can be used to produce a planning level estimate of the cost of the alternative. The approach is much easier to use than the cost element approach, and where a large set of construction projects have been built by a given firm or agency, accurate planning level estimates can be developed.

Unfortunately, no firm or agency has designed every possible project and the need therefore arises for extrapolation of the estimates. How much more does it cost if the alternative has 50% or more rock excavation than its predecessor? Or what if the pump station has to supply 200 ft of head at 30 cfs and the files only contain designs for 80 ft at 25 cfs and 60 ft at 39 cfs? It is this type of question that limits the applicability of historical data to very standard projects (e.g., 6-in. water distribution mains) or the role of a "double-check" on a cost estimate (an estimate that shows a 42 in. pipe costing less than a newly installed 36 in. pipe should be questioned).

It is also important in a planning study to examine a broad range of alternatives. Use of existing data tends to restrict the analysis to the type of project with which the firm or agency already has a hefty file of cost data. This was undesirable for the Corps' Urban Studies. For example, land treatment of wastewater is given a great deal of consideration in Urban Studies, yet very few firms have designed land treatment systems.

Parametric Methods

Parametric costing involves development of a function relating the cost to a small number of design parameters which are known by the planner. Ideally, this can be represented by a function of a single variable. A function such as

Average Annual Cost of Wellfield
= f(Safe Yield) (1)

would save the planner a great deal of work. Of course the function given above would be virtually useless since it does not account for geologic formation, depth of wells, groundwater depth, drawdown depth, piping and pumping equipment, labor cost, interest rate, design life, peak/average flow ratio, number of wells, and power cost. Any attempt to apply multiple regression techniques to this list of variables would result in failure to arrive at an acceptable correlation. Nevertheless, curve fitting for a small number of independent variables could provide accurate estimates. For example

Installed cost of gate valve
= f(pipe diameter) (2)

can produce an acceptable estimate for planning studies.

Parametric costing methods have been developed for wastewater treatment plants (Patterson and Banker, 1971; Van Note et al., 1975), water treatment (Gumerman, Culp and Hansen, 1978) and general water resources studies (Dawes, 1970; Koenig, 1966; U. S. Department of Interior, 1959). Most of the cost functions are based on regression analysis of existing data for actual construction while some of the more recent efforts (Gumerman, Culp and Hansen, 1978; Van Note et al., 1975) are based on curve-fitting of typical designs for an array of sizes and design parameters. The accuracy of these cost estimating methods depends on the selection of the independent and dependent variables. For example, when Dawes (1970) correlates reservoir costs with storage volume, the correlation is poor since the reservoir costs depend on a number of variables (e.g., embankment height, spillway capacity, clearing cost), which are not closely related to storage volume.

The user of parametric costs must be careful to understand the assumptions used in developing the cost function, and the range over which the function is valid. For example, Van Note et al. (1971) gives the cost of a primary clarifier as a function of flow, assuming an influent suspended solids of 230 mg/ℓ and a surface loading rate of 800 gpd/ft^2. The user has no way of modifying the costs if he uses a different design (a similar problem as in the case of using historical data).

The key to using the parametric approach is to divide the project costs into costs of separate items (cost elements) for which rational cost functions can be derived. This implies the need for a synthesis of the cost element and parametric costing approaches.

SELECTION OF METHOD

A "modified cost element" approach to cost estimating has evolved from this synthesis of the parametric and cost element approaches. In this approach, the project is divided into larger cost items--for example, the modified cost element reinforced concrete is used in place of the cost elements concrete, rebar, formwork, etc. The use of these modified cost items greatly simplifies the work without seriously affecting the accuracy of the estimates.

This modified cost element method is being used in two computer-aided costing methods developed by the U. S. Army Engineer Waterways Experiment Station for Corps Urban Studies. These tools are CAPDET (Computer Assisted Procedure for Design and Evaluation of Alternative Wastewater Treatment Systems)(Department of the Army, 1978), and MAPS (Methodology for Areawide Planning Studies)(Department of the Army, 1979). CAPDET produces cost estimates for wastewater treatment plants and uses a fairly large number of modified cost elements for each unit process. MAPS is a multipurpose program which produces cost estimates for a large array of "facilities" (e.g., open channels, pipelines) at a much lower level of detail. CAPDET has much more in common with the cost element approach using unit prices, while MAPS uses very large cost elements with more reliance on cost curves.

MAPS Method

The costs of a facility or project in MAPS is the sum of the costs of the elements making up the facility. These elements may be further divided into smaller elements until a simple function of one or two variables can be used. This is shown schematically below in Fig. 1.

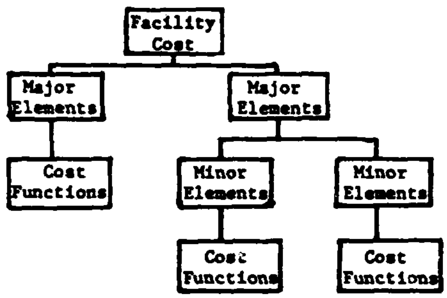

Fig. 1. Schematic for MAPS costing

Very simple cost functions are used. The two most commonly used forms are

$$\text{Cost} = \text{Quantity} \times \text{Unit Price} \tag{3}$$

or

$$\text{Cost} = a\ (\text{Quantity})^{b} \tag{4}$$

which correspond to straight lines on arithmatic and log-log graph paper, respectively. If a straight line cannot be used to represent the cost data, a piecewise curvefit is used instead of a complicated polynomial, which may have undesirable critical points and points of inflection. For example, the data on Fig. 2 would be represented by

$$C = \begin{cases} a_1 x^{b_1}, & x \leq p \\ a_2 x^{b_2}, & x > p \end{cases} \tag{5}$$

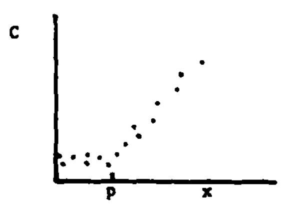

Fig. 2. Typical cost data

instead of

$$C = ax^3 + bx^2 + cx + d \tag{6}$$

In many cases, the parameters known by the planner (e.g., population, safe yield) are not the independent variable (i.e. design parameter) used in the cost function (e.g., pipe diameter, normal depth). A preliminary design routine is required to convert the planner's parameters into the design parameter. In some cases, this procedure is fairly complicated as in the case of determining normal depth of a trapezoidal channel, which requires an iterative solution. These steps, which can be added to the block labeled "Cost Function" in Fig. 1, are given in Fig. 3.

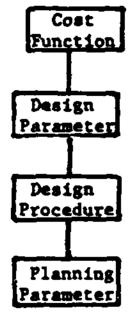

Fig. 3. Schematic of MAPS design

The modules for which MAPS costing procedures exist include

1. force mains,
2. gravity mains,
3. open channels,
4. pipelines,
5. pump stations,
6. reservoirs,
7. storage basins/tanks,
8. tunnels,
9. water treatment plants,
10. wellfields.

CAPDET Method

CAPDET uses a similar method to MAPS. Since it is used only for wastewater treatment, the level of detail is more consistent for CAPDET (i.e., major elements correspond to unit processes - minor elements correspond to structural, mechanical equipment, etc.). All cost functions in CAPDET are of the form

$$\text{Cost} = \text{Quantity} \times \text{Unit Price} \tag{7}$$

where the unit price is itself a function of the quantity or size in many cases. The unit price of an item of size z is calculated from the price of a standard size item(s) using

$$\text{Unit Price}_z = \text{Unit Price}_s \times f(z/s) \tag{8}$$

CAPDET uses a two-step design procedure. In the first, the volumes and areas of tanks and loading on mechanical equipment are calculated.

In the second step, these amounts are converted into quantities used for the cost functions such as cubic yards of concrete and feet of piping.

Computerization

A question that arose early in this work was whether to computerize the cost estimating methods. Eilers and Smith (1973) of the U.S. Environmental Protection Agency (EPA) had computerized the Patterson and Banker (1971) wastewater treatment cost curves. Schonbok (1978) of the Bureau of Reclamation maintains a computer program for construction cost of some types of pipe, large pump stations and tunnels. Several private firms (e.g., the ICARUS Corp. (Brand and Crerar, 1974; Epstein, 1974), and the U. S. Department of Transportation (Wheby and Cikanek, 1973)) have produced computer programs for specific types of projects.

Because of the large number of calculations required to perform the cost estimates, CAPDET and MAPS were developed as computerized procedures. Nevertheless, documentation exists for both programs to enable those intent on hand calculating the costs to follow a step-by-step procedure.

Default Data

There are a large number of independent variables that need to be specified to arrive at the cost estimates. Such parameters as depth of cover for pipes, loading rates for filters, freeboard and side slope for open channels are examples of variables that must be specified in order to perform the design and costing calculations. In many cases, the planner does not know or does not want to determine these inputs, but would rather use some typical value. The programs contain a set of these typical values, called "default data." These data are used by the program whenever the user does not specify the values.

Default values exist for variables such as side wall depth of clarifiers in water treatment plants (10 ft), lining type and thickness for canals (6 in. concrete) and type of pump for wells (vertical turbine). Of course, default data cannot be specified for certain inputs such as flow and length for pipelines. With this default data, the user can develop cost estimates for typical facilities early in a study and then refine these estimates as he determines more information in the later stages of the planning process.

EXAMPLES

Wellfields

The cost procedure used by MAPS can best be illustrated by some examples. The wellfield cost estimating routine can be used to determine the cost of a wellfield consisting of any number of wells, drilled to roughly the same depth, with a uniform static groundwater elevation and staged over a number of years, which is less than the design life of a given well. The wellfield module of MAPS can account for the effect on the cost of

1. flow (peak and average),
2. drilled depth,
3. static groundwater level,
4. drawdown,
5. number of wells,
6. distance between wells,
7. time over which construction staged,
8. cost of power,
9. interest rate,
10. inflation rate,
11. ENR index value,
12. pressure required in surface pipes,
13. housing for wells,
14. test wells,
15. year construction initiated,
16. design life,
17. type of pump,
18. type of material,
19. O&M labor cost,
20. land cost,
21. efficiency.

Default values exist for most of the above variables. The relationship of these variables in calculating the average annual cost of a wellfield is shown schematically in Fig. 4. This figure is typical of charts used by program developers in preparing the costing procedure.

The existing MAPS wellfield module is actually the second generation of the module. The initial version calculated the cost of the wells alone and it was necessary for the user to calculate the piping and energy costs of the wellfield as a whole. The program now performs these calculations automatically.

Pipeline

The pipeline design module actually consists of a force main module and pump station module, which can be run separately or together using the pipeline option. Given the flow, distance, and change in elevation, the program designs and costs a pipeline with pumping stations for an array of pipe sizes. Of course, the user can override all of the default values and specify such parameters as depth of cover, number of pump stations, type of pipe, valves, bends, hydrants, to name just a few. (The list is longer than for wellfields, above.)

The calculations required to make a pipeline design and cost analysis by hand would take several days. Using MAPS, the planner can learn to use that portion of the program and generate the cost estimates in less than a day. The logic used in the routine is very simple, but the number of calculations is large, so the computer becomes necessary in such a case. The logic of the pipeline module is shown in Fig. 5.

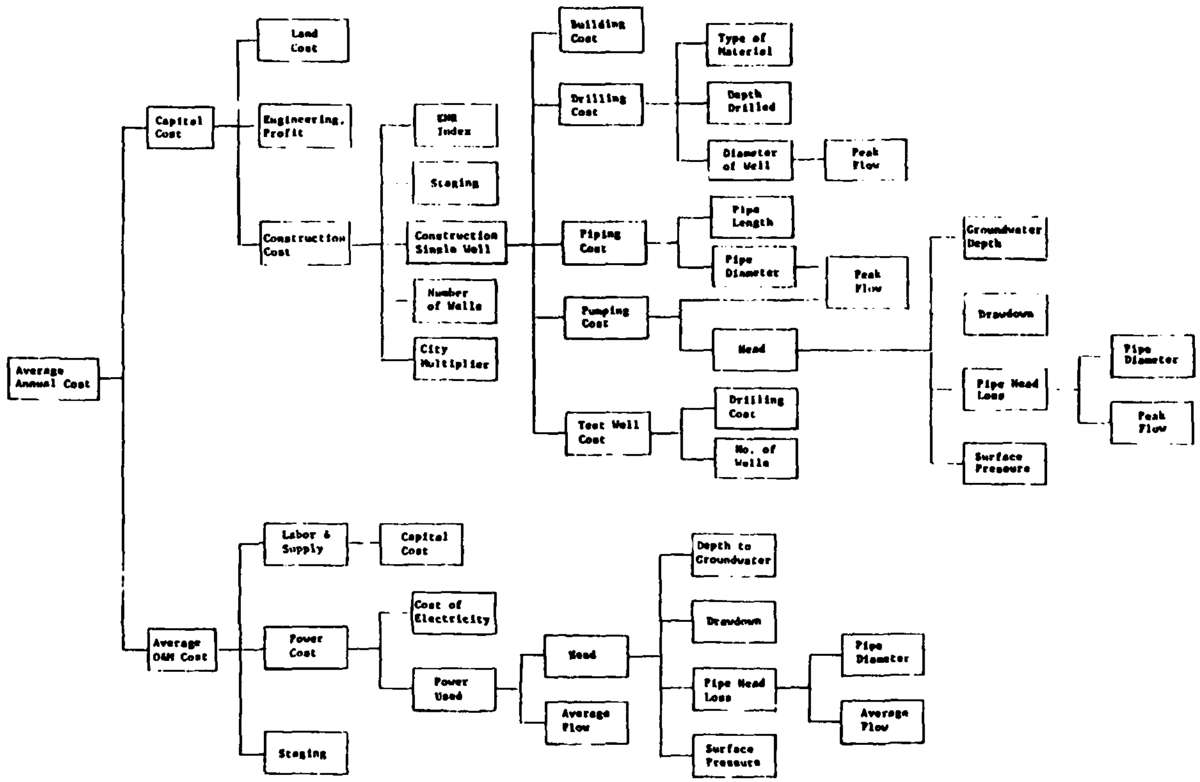

Fig. 4. Schematic of MAPS wellfield module

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VERIFICATION OF COST ESTIMATING PROCEDURES FOR MAPS COMPUTER PR--ETC(U)
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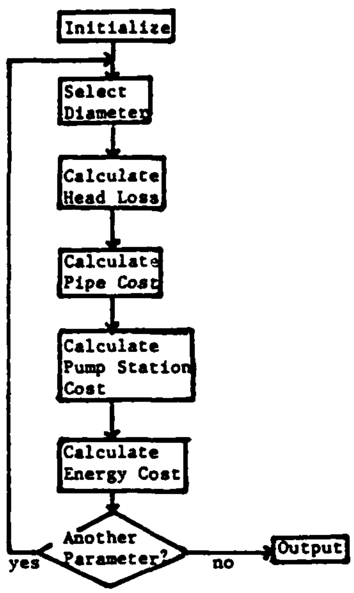

Fig. 5. Pipeline module logic

APPLICATION

The MAPS program has thus far been used primarily in the water supply portion of *Urban* Studies. MAPS was a key component of the water supply task in both the Nashville (Corey, et al., 1978) and Salinas-Monterey (Walski and Gibson, 1979) Urban Studies. It is currently being used by Corps offices for a number of Urban Studies, specially authorized water supply studies, and other studies. The costs have been compared with more detailed costing procedures and have proved to be of acceptable accuracy, especially considering the limited amount of effort required.

The CAPDET program is currently being used by the USEPA and consulting firms to develop cost estimates for wastewater treatment plants, especially for the 201 planning process. The costs produced by the program have proven to be extremely accurate.

The major danger in using these programs is the possibility of applying them to projects which are significantly different than the projects for which they were developed. For example, using the MAPS pipeline module to cost a pipeline being laid under water, or using CAPDET to design a package treatment plant to handle 50 mgd would result in poor estimates. The users of the program are warned that they must examine the procedure used to be certain that it is applicable to their project.

CONCLUSION

The modified cost element approach to cost estimating as used to different extents in the CAPDET and MAPS programs can produce costs of sufficient accuracy for planning studies with a minimum of time and effort when used properly. The methods used in these programs can be applied to a wide array of other cost estimating problems. Such methods generally require a computerized solution because of the large number of equations used.

ACKNOWLEDGEMENTS

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APPENDIX C: CHANGES TO USER'S GUIDE AND DOCUMENTATION

CHANGES TO USER'S GUIDE AND DOCUMENTATION

Introduction

1. As a result of the verification study, modifications have been made to the MAPS program. The changes include new keywords and new or revised equations. This section provides a summary of these changes to the program and is included to supplement Parts 1 and 2 of Engineer Manual EM 1110-2-502. Definitions are given only for new variables.

Force Mains

Culture multipliers

2. New multipliers to correct cost for type of terrain are as follows:

$$\text{MULT} = \begin{cases} 0.67 & \text{open country} \\ 0.91 & \left\{\begin{array}{l}\text{new residential} \\ \text{sparse residential}\end{array}\right. \\ 1.08 & \text{dense residential} \\ 1.20 & \left\{\begin{array}{l}\text{commercial} \\ \text{central city}\end{array}\right. \end{cases}$$

If the type of terrain is nót specified, MAPS assumes that the multiplier is equal to 1.0. If the sum of C_i (percentage of length in each type of area) is less than 100 percent, the remainder of the pipe is assumed to have a culture multiplier of 1.0.

Steel reinforcement

3. The weight of reinforcing steel required must be determined for reinforced concrete pipe, prestressed cylinder pipe, and pretensioned concrete cylinder pipe. The equations for each type of pipe have been *modified based on a better curve fit of the original data.*

a. Reinforced concrete pipe.

$$\text{REINF} = \begin{cases} 1.609 \times 10^{-3}\text{DIAM}^{2.01}\text{PRES}^{0.5873} & \text{, laying condition A} \\ 3.444 \times 10^{-3}\text{DIAM}^{2.01}\text{PRES}^{0.4449} & \text{, laying condition B} \\ 6.9582 \times 10^{-3}\text{DIAM}^{2.00}\text{PRES}^{0.3311} & \text{, laying condition C} \\ 9.0815 \times 10^{-3}\text{DIAM}^{2.05}\text{PRES}^{0.2652} & \text{, laying condition D} \end{cases}$$

b. Prestressed cylinder pipe.

$$\text{REINF} = \begin{cases} 1.678 \times 10^{-5}\text{DIAM}^{2.28}\text{PRES}^{0.8537} & \text{, laying condition A} \\ 2.513 \times 10^{-5}\text{DIAM}^{2.23}\text{PRES}^{0.8541} & \text{, laying condition B} \\ 2.684 \times 10^{-5}\text{DIAM}^{2.30}\text{PRES}^{0.8152} & \text{, laying condition C} \\ 4.084 \times 10^{-5}\text{DIAM}^{2.51}\text{PRES}^{0.6114} & \text{, laying condition D} \end{cases}$$

c. Pretensioned concrete cylinder pipe.

$$\text{REINF} = \begin{cases} 2.2233 \times 10^{-7}\text{DIAM}^{2.27}\text{PRES}^{1.8028} & \text{, laying condition A} \\ 3.4688 \times 10^{-6}\text{DIAM}^{1.91}\text{PRES}^{1.5349} & \text{, laying condition B} \\ 2.4413 \times 10^{-6}\text{DIAM}^{2.06}\text{PRES}^{1.5237} & \text{, laying condition C} \\ 4.6076 \times 10^{-6}\text{DIAM}^{2.04}\text{PRES}^{1.4402} & \text{, laying condition D} \end{cases}$$

Trench bottom width

4. If the user does not specify the width of the trench in excess of the pipe diameter, the total bottom width is calculated as

$$\text{BWIDTH} = \begin{cases} \text{DIAM}/12 + 1.25 & , \ \text{DIAM} < 24 \text{ in.} \\ 0.3023*\text{DIAM}^{0.8163} & , \ \text{DIAM} \geq 24 \text{ in.} \end{cases}$$

Excavation and backfill

5. Updated unit prices for excavation and backfill are given as

$$\text{UCCEX} = \begin{cases} 1.72 & , \text{ if } \ \text{TDEPTH} \leq 7 \text{ ft} \ \text{ and } \ \text{OCCEX} \leq 0 \\ 8.01*\text{TDEPTH}^{-0.7906} & , \text{ if } \ 7 < \text{TDEPTH} < 20 \text{ ft} \ \text{ and } \ \text{OCCEX} \leq 0 \\ 0.75 & , \text{ if } \ \text{TDEPTH} \geq 20 \text{ ft} \ \text{ and } \ \text{OCCEX} \leq 0 \end{cases}$$

$$\text{UCBAC} = 0.61 \ , \ \text{ if } \ \text{OCBAC} \leq 0$$

Pump Stations

New keywords

6. Additional keywords that may be used in the pump station module are listed below.

WATER PUMPING

WASTEWATER PUMPING

NO BUILDING

PUMPING UNITS PER STATION XX.X

Contingencies

7. Total cost for each stage of construction is the sum of the cost of mechanical equipment, structure, electrical accessories, etc., with a contingency factor of 30 percent. If the type of station is not specified, the program assumes that the station is for treated or raw water pumping (not wastewater). Stages 1 and 2 construction costs are given as

CC1 = 1.3*[CMECH(1) + CSTRUC + CELEC(1) + CSWIT + CWET + CMISC(1)]

CC2 = 1.3*[CMECH(2) + CELEC(2) + CMISC(2)]

Number of pumping units

8. The number of units per station may be specified by the user or assumed by the program based on peak flow rate.

$$NP = \begin{cases} NPU, & \text{if } NPU > 0 \\ 2, & \text{if } QMAX < 15 \text{ mgd and } NPU \leq 0 \\ 3, & \text{if } 15 \leq QMAX < 46 \text{ mgd and } NPU \leq 0 \\ 4, & \text{if } 46 \leq QMAX < 140 \text{ mgd and } NPU \leq 0 \\ 5, & \text{if } 140 \leq QMAX < 400 \text{ mgd and } NPU \leq 0 \\ 6, & \text{if } QMAX \geq 400 \text{ mgd and } NPU \leq 0 \end{cases}$$

where

NP = number of pumping units

NPU = number of units specified by user

QMAX = maximum flow, mgd

Structure cost

9. In addition to costs for simple and improved structures, the user may specify no housing (CSTRUC = 0). For small pump stations (QMAX < 5.0 mgd), structure costs are now given as

$$\text{CSTRUC} = \text{CFAC} * 9846 * \text{QMAXS}^{0.35}$$

Wet well costs

10. Updated costs functions for wet wells are given as

$$\text{CWET} = \begin{cases} \text{CFAC} * 220{,}053 * \text{WVOL}^{0.421}, & \text{WVOL} \leq 0.15 \text{ mg} \\ \text{CFAC} * 386{,}221 * \text{WVOL}^{0.724}, & \text{WVOL} > 0.15 \text{ mg} \end{cases}$$

where

CFAC = ENR/2877

Mechanical equipment cost

11. The cost of mechanical equipment for large pump stations (QMAX > 5.0 mgd) can be given as

$$\text{CMECH} = 216 * H^{0.4} (\text{QMAX} * 1.54)^{0.935} * \text{CFAC} * 2.67 * \text{XTYP}$$

where

$$\text{XTYP} = \begin{cases} 1.0, & \text{raw or treated water pumping} \\ 1.4, & \text{wastewater pumping} \end{cases}$$

Mechanical equipment cost for small (QMAX < 5.0 mgd) pump stations is given by

$$\text{CMECH} = \begin{cases} \text{CFAC} * 1316 * \text{QMAX}^{0.678} H^{0.561} * \text{XTYP}, & 0.72 \leq \text{QMAX} < 5 \\ \text{CFAC} * 2205 * \text{QMAX}^{0.264} H^{0.438} * \text{XTYP}, & \text{QMAX} < 0.72 \end{cases}$$

where

$$XTYP = \begin{cases} 1.0 \text{ , raw or treated water pumping} \\ 1.2 \text{ , wastewater pumping} \end{cases}$$

Electrical equipment cost

12. The cost of electrical equipment is related to head, flow, and number of pumping units as

$$CELEC = CFAC * 732\ QMAX^{0.585}\ H^{0.472}\ NP^{0.446}$$

where

NP = the number of pumping units per station

Miscellaneous costs

13. The cost of miscellaneous equipment is given as

$$CMISC = CFAC * 15,184 * QMAX^{0.457}\ NP^{1.01}$$

Wellfields

New keywords

14. Additional keywords available to the user in the wellfield module are listed below:

DIAMETER OF WELL	XX.X INCHES
DISTANCE	XX.X MILES
TUBULAR WELL (default)	
GRAVEL-PACK WELL	
STANDARD CONTROLS	
SOPHISTICATED CONTROLS	
NO CONTROLS (default)	
SIMPLE STRUCTURE (default)	
IMPROVED STRUCTURE	
CONTROL SYSTEM COST OVERRIDE	XX.X DOLLARS

Single well construction cost

15. The total construction cost for a single well has been expanded to include the cost of control equipment.

$$CIND = CDRILL + CPIPE + CPUMP + CHOUSE + CTEST + CCONT$$

where

CCONT = cost of control equipment, $

Drilling costs

16. Two types of wells may be specified in MAPS for an unconsolidated aquifer--tubular and gravel pack. The existing cost equations for drilling in "unconsolidated aquifers" apply to gravel-pack wells, and drilling costs for tubular wells are given as

$$CDR = 142 * DRDEP^{0.33}\ DIAMW^{0.76}$$

Piping costs

17. If the wellfield is assumed to have a linear arrangement, piping is sized for cumulative flow progressing along the pipeline. Costs for piping are given by

$$CPIPE = CPIPE_1 + CPIPE_2 + \ldots + CPIPE_n$$

where

n = number of wells minus one

$$CPIPE_i = CUNIT_i * DIST * 5280 * CITY * ENR/2680$$

where

DIST = average distance between wells, miles

$CUNIT_i$ = unit price of pipe, \$/ft

Unit prices are based on PVC pipe (for DIAM < 10 in.) and ductile iron pipe (DIAM $\geq$ 10 in.), and are given below:

CUNIT \$/ft	DIAMP in.
3.44	4
4.82	6
6.16	8
9.23	10
14.31	12
16.00	14
18.21	16
21.45	18
24.79	20
32.29	24
38.00	27
44.04	30
56.79	36
71.00	42
84.90	48
101.50	54
116.07	60
135.00	66
153.00	72

The pipe is sized to flow at 5 ft/sec at flow rate QMI . The diameter is, therefore,

$$DP_i = \sqrt{\frac{QMI_i * 1.54 * 4}{5 * \Pi}} * 12$$

$$= 7.51\sqrt{QMI_i}$$

where

$$QMI_{i+1} = QMI_i * (i + 1)$$

The nominal pipe size $DIAMP_i$ is the next largest pipe size listed in the table.

Head loss

18. Head loss at peak flow in connecting pipes for wells in a linear arrangement may be approximated by

$$PLOSSM = DIST * (N - 1) * 5280 * 0.002 \ , \quad DIST > 0$$

or average flow, head loss is given as

$$PLOSSA = DIST * (N - 1) * 5280 * 0.001 \ , \quad DIST > 0$$

Housing costs

19. Costs for well housing may be given by

$$CHOUSE = \begin{cases} 0 & \text{, no housing} \\ ENR/2877 * 9846 * QMI^{0.35} & \text{, simple structure} \\ ENR/2877 * 25{,}000 * QMI^{0.35} & \text{, improved structure} \end{cases}$$

Control systems

20. The cost of control systems may be given by

$$CCONT = \begin{cases} 0 & \text{, no controls and } UCONT \le 0 \\ 5{,}000 * ENR/3372 & \text{, standard controls and } UCONT \le 0 \\ 10{,}000 * ENR/3372 & \text{, sophisticated controls and } UCONT \le 0 \\ UCONT & \text{, } UCONT > 0 \end{cases}$$

where

CCONT = cost of control systems per well, $

UCONT = control system cost override (cost per well), $

Storage Tanks

New keywords

21. The capability of specifying BURIED CONCRETE TANK has been added to the list of keywords for the storage tank module.

Buried concrete tanks

22. The costs of buried concrete tanks can be given as

$$CC = CFAC * 315{,}528 * VOL^{0.753}$$

where

$CFAC = ENR/2877$

Earth spillway costs

23. Updated costs for nonconcrete overflow sections are given below.

$$CSPILE = CFAC * 0.962 \left[(HMAX - 5) * QSPIL^{0.5}\right]^{1.48}$$

Embankment protection

24. The cost of embankment protection is based on the area of the upstream and downstream faces calculated for the height of cover on the faces. This area is calculated as

$$UFAREA = HCOVU * (CRLEN - SPLSEC) * \left[1 - \frac{1}{2}\left(\frac{HCOVU}{HMAX\sqrt{1 + SLU^2}}\right)\right]$$

$$DFAREA = HCOVD * (CRLEN - SPLSEC) * \left[1 - \frac{1}{2}\left(\frac{HCOVD}{HMAX\sqrt{1 + SLD^2}}\right)\right]$$

Open Channels

New keywords

25. The user may specify the percent accuracy desired for convergence (difference between predicted flow based on normal depth and actual flow) in calculating normal depth in an open channel by entering PERCENT ACCURACY XX.X PERCENT. The default value assumed by MAPS is 1.0 percent.

Siphon costs

26. The modified procedure for calculating the cost of siphons and associated transitions is given by

$$CSIPH_i = XFAC1 * COPIP$$

$$COPIP = \begin{cases} [-600 + 101 * DIAM + 0.887 * SIPH(I,2)] * SIPH(I,1) * NBARR \; , \\ \quad DIAM > 7.5 \text{ ft} \\ (17.65 * DIAM^{0.70} DEPTH^{0.132}) * SIPH\ (I,1) \; , \quad DIAM \leq 3.5 \text{ ft} \\ (6.73 * DIAM^{1.49} DEPTH^{0.132}) * SIPH\ (I,1) \; , \; 3.5 < DIAM \leq 7.5 \text{ ft} \end{cases}$$

The diameter of the pipe is calculated by

$$DIAM = \begin{cases} \left(0.64\ Q^{0.863}\right)^{1/2} & , \quad 1 < Q \leq 15 \text{ cfs} \\ \left(1.14\ Q^{0.647}\right)^{1/2} & , \quad 15 < Q \leq 850 \text{ cfs} \\ (0.13\ Q)^{1/2} & , \quad 850 < Q \leq 5000 \text{ cfs} \\ [0.13 \cdot Q/(NBARR)]^{1/2} & , \quad Q > 5000 \text{ cfs} \end{cases}$$

Drop structures

27. The volume of concrete required for a drop structure may be determined by

$$VOLC = \begin{cases} 1.11 * DROP^{0.384}\ Q^{0.532} & , \quad Q < 80 \text{ cfs} \\ 0.047 * Q^{0.131} + 2.91 * DROP^{0.757} & , \quad Q \geq 80 \text{ cfs} \end{cases}$$

Radial gates

28. The cost of a radial gate is given by

$$\mathrm{CRAD}_i = \begin{cases} \mathrm{XFAC} * 926.77 * \mathrm{AREARG}_i^{0.608}, & \mathrm{AREARG} \le 175\ \mathrm{ft}^2 \\ \mathrm{XFAC} * 47.81 * \mathrm{AREARG}_i^{1.18}, & \mathrm{AREARG} > 175\ \mathrm{ft}^2 \end{cases}$$

Calculation of normal depth

29. The normal depth of flow for a channel is determined using an iterative solution. The flow in the channel is known; therefore, a normal depth is determined which corresponds to the flow, defined as

$$Q = \frac{K' \, b^{8/3} \, S_o^{1/2}}{n} \tag{C1}$$

where

Q = flow, cfs

b = bottom width of channel, ft

S_o = slope of channel bed

n = Manning's n

K' = conveyance for trapezoidal channel

and

$$K' = \frac{1.49 \, (1 + my/b)^{5/3}}{\left[1 + 2\sqrt{1 + m^2} \, (y/b)\right]^{2/3}} \; \frac{y}{b}^{5/3}$$

where

m = side slope of channel (cot θ)

θ = side slope angle, degrees from horizontal

y = normal depth of flow, ft

b = bottom width of channel, ft

An initial value for depth is selected y_i, and flow is computed and compared to the actual flow. A new value for depth is then calculated using a modification of the Newton-Raphson method:

$$y_{i+1} = y_i - \frac{f(y_i) - Q_a}{f'(y_i) - Q_a} \tag{C2}$$

where

$$Q_a = \text{actual flow in channel, cfs}$$

$$f'(y_i) = Q_i' = \frac{f(y_i) - f(y_{i-1})}{y_i - y_{i-1}}$$

$f(y_i) = Q_i$ as defined in Equation C1

The iterations continue in this manner until the following criterion is met:

$$-\Delta < f(y_i) - Q_a < \Delta$$

where

$$\Delta = PERC * Q_a/100$$

PERC = desired accuracy, percent

$$= \begin{cases} UPERC, & UPERC > 0 \\ 1.0, & UPERC \le 0 \end{cases}$$

UPERC = accuracy specified by user, percent

30. In the open channel module, Equation C1 takes the form:

$$FM = \frac{TCONV * B^{8/3} SLOPE^{1/2}}{XMANN} - Q$$

where

FM = flow, cfs

B = bottom width, ft

SLOPE = slope of channel bed

XMANN = Manning's n, default = 0.014 for portland cement concrete

Q = actual channel flow, cfs

TCONV = conveyence

and

$$TCONV = \frac{A1 * (1 + A2 * YM)^{1.667}}{(1 + A3 * YM)^{0.667}} * YM^{1.667}$$

where

$$A1 = 1.49/B^{1.667}$$

$$A2 = C/B$$

$$A3 = 2/B * \sqrt{1 + C^2}$$

YM = depth, ft

C = side slope of channel, C:1

= cotangent of side slope angle

THETA = side slope, degrees from horizontal

FM1 and TCONV1 are expressed in a similar manner, corresponding to YM1 . Equation C2 will take the form:

$$YM2 = YM1 - FM1/DFM$$

where

$$DFM = \frac{FM1 - FM}{YM1 - YM}$$

If FM1 (difference between flow calculated based on normal depth and actual channel flow) is within 1 percent (or percentage specified by user) of the actual flow:

$$YN = YM$$

If not, the process is repeated for the following new values:

$$YM = YM1$$

$$YM1 = YM2$$

$$FM = FM1$$

Tunnels

Unconfined compressive strength

31. Excavation costs for machine-bored tunnels are calculated using equations given in MAPS documentation (EM 1110-2-502 Part 2). The value of unconfined compressive strength dictates which set of equations is applicable to a particular situation. New limits are given below for these limits:

Set of Equations	Unconfined Compressive Strength, psi	
	Old Limits	New Limits
(1)	$\leq 5,000$	$\leq 7,500$
(2)	5,000-10,000	7,500-15,000
(3)	10,000-20,000	15,000-30,000
(4)	20,000-40,000	30,000-40,000

These new limits are also applicable to the equations for determining dewatering costs.

Rock quality designation (RQD)

32. The MAPS equations used to determine excavation and dewatering costs in both drill and blast tunnels and machine-bored tunnels are further subdivided according to the range of the RQD index. New limits are listed below:

RQD	
Old Limits	New Limits
$40 \leq RQD < 60$	$40 \leq RQD < 50$
$60 \leq RQD < 80$	$50 \leq RQD < 70$
$80 \leq RQD < 100$	$70 \leq RQD < 90$
$RQD = 100$	$90 \leq RQD \leq 100$

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Lindsey, Anita K.
Verification of cost estimating procedures for MAPS computer program / by Anita K. Lindsey, Thomas M. Walski (Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1982.
111 p. in various pagings ; ill. ; 27 cm. -- (Technical report ; EL-82-3)
Cover title.
"May 1982."
Final report.
"Prepared for Office, Chief of Engineers, U.S. Army under Contract No. DACW39-81-M-0722, Work Unit 31572."
Bibliography: p. 79-80.

1. Computer programs. 2. Cost effectiveness. 3. MAPS (Computer programs). 4. Water resources development. I. Walski, Thomas M. II. United States.

Lindsey, Anita K.
Verification of cost estimating procedures : ... 1982.
(Card 2)

Army. Corps of Engineers. Office of the Chief of Engineers. III. U.S. Army Engineer Waterways Experiment Station. Environmental Laboratory. IV. Title V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; EL-82-3.
TA7.W34 no.EL-82-3